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WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
THIRD FLOOR, PARK SQUARE BUILDING
31 ST. JAMES AVENUE
BOSTON 16, MASS.

NEBGD 10 October 1945

MEMORANDUM TO: Chief, Engineering Division

CONCERNING: Definite Project Report, North Andover and Lawrence, Massachusetts - Local Protection

1. Reference is made to paragraph 2 in the 5th Indorsement, dated 3 October 1945, on the above subject, from the New England Division to the Office, Chief of Engineers. The following comments are made relative to the discussion on the proposed capacity of the pumps for the North Andover Pumping Station:

(a) The capacity of the existing drainage system is 20 c.f.s.

(b) The capacity of the existing trunk sewer is 6 c.f.s., but it is estimated that due to the small area served by the sewerage system, the maximum flow will be 2 c.f.s.

(c) With the proposed drainage interceptors the total drainage capacity will be 34 c.f.s. and, combined with the sewage flow of 2 c.f.s., the maximum inflow to the pumping station will be 36 c.f.s. It should be noted that emergency outlets through the walls are provided for storm run-off in excess of the drain capacity to prevent ponding within the walled area.

(d) The pumping capacity in the initial project report submitted 22 May 1944 proposed pumping capacity of ~~40~~ 50 c.f.s. in order to provide an emergency pumping unit. An indorsement from the Office, Chief of Engineers, dated 9 September 1944, relative to the Nashua Definite Project Report, noted that, "It is considered that a pump capacity greater than the capacity of the existing sewers is not warranted". Consequently, in reviewing the design of the proposed North Andover Pumping Station, this recommendation was followed and the pump capacity was reduced to 36 c.f.s.

(e) The indorsement deducts 6 c.f.s. for sewage flow and obtains a pump capacity of 30 c.f.s. to handle storm run-off instead of the noted 34 c.f.s. and as discussed in the project report.

(f) The 0.85 inch rainfall is derived from the assumed run-off of 30 c.f.s. and a run-off coefficient of 0.45. It should be noted that the 0.85 inches is a rate of rainfall expressed in inches per hour even though the assumed storm lasted only 30 minutes. The volume of rain in that 30 minute period was 0.43 inches, but the rate was 0.85 inches per hour. The mentioned 0.30 inches per hour is not understood.

(g) The frequency of a storm with a 30 minute intensity of 0.85 inches per hour is approximately once every 5 months. It is believed that the indorsements method of expressing frequency in "percent chance of occurrence in any one year" is misleading.

(h) The New England Division "considers a storm of a one (1) percent chance of occurrence in any one year to be the minimum value upon which to base the design of the station". Conceivably, this would be a storm with a frequency of once in 100 years, but a frequency of once a year is actually intended. Selecting pump capacity on the basis of storm frequency gives the impression that the pumps will be utilized to capacity every year. However, emphasis should be made on the improbability that the one year storm will synchronize with the flood stage in the river, for the storm and the flood stage must occur concurrently to require full use of the pumps. Exact mathematical analysis of the probability of simultaneous occurrence of storm and flood stage is impossible but Plates 11, 11a, 11d, in the Project Report show graphically the frequency relationship of each factor separately.

(i) The 2nd Indorsement from the Office, Chief of Engineers, dated 31 August 1944, recommended reviewing the proposed capacity of the pumps for the Lawrence Pumping Station, believing that the size of the pumps could be reduced. No mention was made of the selected pumps for the North Andover Pumping Station, but in view of the noted recommendation concerning the Nashua pumps, it was concluded that the same policy should be applied to the pumps for all stations.

E. F. Childs
Engineer

DEFINITE PROJECT REPORT

on

NORTH ANDOVER AND LAWRENCE, MASS.,

LOCAL PROTECTION WORKS, MERRIMACK RIVER BASIN, MASS.

Prepared in the U. S. Engineer Office,

Boston, Massachusetts, dated May 1944
revised Dec. 1944

Approved by the Chief of Engineers _____ 194_

E.D. File 800.5(Shawsheen River)

REVISIONS

(See following page)

(This revised report supersedes report dated May 1944)

Definite Project Report

NORTH ANDOVER AND LAWRENCE, MASSACHUSETTS, LOCAL PROTECTION

Merrimack River Basin Flood Control

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Corps of Engineers, U. S. Army

U. S. Engineer Office Boston, Massachusetts

May 1944

(Revised 23 December 1944)

WAR DEPARTMENT
United States Engineer Office
3d Floor, Park Square Bldg.
31 St. James Avenue
Boston 16, Mass.

May 1944
(Rev. 23 Dec. 1944)

Subject: Definite Project Report for North Andover and Lawrence, Massachusetts, Local Protection

To: The Chief of Engineers, U. S. Army, Washington, D. C., Through the Division Engineer, New England Division, Boston, Massachusetts.

1. Project Authority.- The projects for local protection at North Andover and Lawrence, Massachusetts, as described herein, are proposed as part of the comprehensive plan for flood control reservoirs and related flood control works for the Merrimack River Basin authorized by the Flood Control Acts approved 22 June 1936 and 28 June 1938. The Flood Control Act of 1938 provides that, "The project for flood control in the Merrimack River Basin as authorized by the Flood Control Act approved 22 June 1936, is modified to provide, in addition to the construction of the system of flood control reservoirs, related flood control works which may be found justified by the Chief of Engineers".

2. Previous Investigations.- The investigations and studies upon which the authorized project for the Merrimack River Basin is based are described in House Document No. 689, 75th Congress, 3rd Session.

a. The local protection work for North Andover was contemplated in 1940 and reference is made to the "Survey Report for Navigation, Flood Control, and Water Power, Merrimack River, Massachusetts and New Hampshire", dated 1 April 1940 and submitted to the Chief of Engineers. This report notes that the local flood protection works were under consideration and presents an approximate estimate of the proposed construction costs. Reference is made also to the "Subproject for North Andover, Merrimack River Basin, Mass.", dated 22 August 1940, submitted to the Chief of Engineers.

b. The City of Lawrence planned to sponsor two projects under the Works Progress Administration in 1939:

(1) The construction of dikes and walls in south Lawrence along the west bank of the Shawsheen River, similar to the layout shown on Plate 5, and

(2) The construction of dikes and walls in west Lawrence along the north bank of the Merrimack River, similar to the plan shown on Plate 12.

Work on the south Lawrence project was not started due to the uncertain conditions of that period. The west Lawrence project was initiated but only partially completed before labor curtailments forced the construction to terminate. The completed portion of the dike affords no protection in its present condition.

3. Location and Description of Areas Affected.-

a. General.- (1) North Andover and south Lawrence.- The proposed projects are located on the banks of the Shawsheen River which forms the boundary between the City of Lawrence and the Town of North Andover. The sites of the dikes and walls are about one-half mile upstream from the confluence of the Shawsheen and Merrimack Rivers which is 27.3 miles above the mouth of the Merrimack River and approximately 24 miles north of Boston, Massachusetts. The area requiring protection in North Andover is a residential development of approximately 25 acres, with an estimated population of about 500. The total valuation is estimated to be $500,000. The area requiring protection in south Lawrence is a residential development of approximately 40 acres with a population of approximately 1000. The total assessed valuation of the south Lawrence area in 1939 was $770,000.

(2) West Lawrence.- The incomplete W.P.A. project in west Lawrence is located on the north bank of the Merrimack River just upstream from the Essex Dam, or approximately 29.2 miles above the mouth of the Merrimack River. The area subject to damage during past floods consists of approximately 125 acres and is thickly developed with commercial establishments and residences. The total assessed valuation in 1939 was $1,760,000.

b. Flood Conditions.- (1) North Andover and south Lawrence.- All floods that cause damage at these two sites are the result of backwater from flood stages on the Merrimack River and are not produced by flood flows on the Shawsheen River. The Shawsheen River has a total drainage area of 73 square miles above its mouth and is a sluggish stream, its flow controlled by numerous ponds and swamps. The greatest flood of record occurred in March 1936 when the areas in North Andover and south Lawrence were flooded to a maximum depth of about 15 feet. The estimated direct losses totalled $280,000 in North Andover and approximately the same in the south Lawrence area to be protected by this proposed project. Parts of both areas were flooded again during the hurricane flood in September 1938.

(2) West Lawrence.- Flood damage in this area is the result of the backwater head produced by the Essex Dam. In the 1936 flood the area was inundated to a maximum depth of about 4 to 5 feet and in 1938 a smaller area was affected with maximum depths ranging from 1 to 2 feet. The 1936 flood caused estimated direct losses of $165,000.

c. Current Flood Control Projects.- The comprehensive plan for flood control in the Merrimack River Basin and its present status is illustrated on Plate 1. Two reservoirs have been completed: (1) Franklin Falls controlling the Pemigewasset River, the principal tributary of the Merrimack River, and (2) Blackwater Reservoir on the Blackwater River, a tributary of the Contoocook River. Other proposed reservoirs for controlling the Contoocook River are Mountain Brook, West Peterboro, Bennington and Beards Brook. The effects of these reservoirs, both existing and proposed, have been considered in determining grades for the walls and dikes to protect the inundated areas. A local protection project has been proposed for Nashua, New Hampshire, at the confluence of the Nashua and Merrimack Rivers. Local protection projects also have been completed at Fitchburg, Lowell and Haverhill, Massachusetts. There are no other flood control works, existing or contemplated, which affect the projects described herein.

4. Definite Project Plan.-

a. Work Proposed.- The areas to be protected and the proposed plans for preventing damage from future floods are shown on Plates 3, 4, 5 and 12. All the areas are low and subject to inundation during the flood stages on the Merrimack River. It is proposed to provide protection similarly in all areas by constructing earth dikes and concrete walls. In North Andover and south Lawrence, it is necessary to provide pumping facilities for the sewage and drainage at each site. More detailed description of the proposed construction in the different areas are as follows:

(1) North Andover.- Approximately 2300 feet of concrete wall and 750 feet of earth dike are proposed in this section. New drains will be installed adjacent to the wall and dike to intercept surface run-off, and a pumping station will be constructed to pump both sewage and storm drainage from the local drainage area during flood periods.

(2) South Lawrence.- Approximately 3250 feet of concrete wall and 570 feet of earth dike are proposed in this section. New drains, interceptors, and diversion pipe will be installed to prevent flooding by backwater from the combined sewage and drainage system. A pumping station will be constructed to discharge the sewage and drainage during flood periods.

(3) West Lawrence.- The work in the west Lawrence area consists of completing the W.P.A. earth dike. The original W.P.A. plans specified an earth dike approximately 2300 feet long on the north bank. About 1000 feet of this dike was completed in the 1940 construction period, but the remainder of the dike requires riprapping, gravel toe drain, topsoil and seeding. The low area east of this earth dike will be protected by a concrete wall extending 350 feet along the river side of the O'Sullivan Park and a low earth dike approximately 375 feet in length. No piping or pumping station is required for this section.

b. Design Flood.- The greatest flood of record on the Merrimack River occurred in March 1936, when the discharge at Lawrence was 174,000 c.f.s. This flood exceeded all previous floods on the Merrimack River in an official record of 90 years and an unofficial record of nearly 200 years by 66,000 c.f.s. and 9 feet in stage. The flood in September 1938 had a peak discharge of about 120,000 c.f.s. and is the second highest flood of record. All flood flows on the Merrimack River are produced by either, or a combination of, heavy rainfall and melting snow in the headwaters of the Merrimack River Basin. The drainage areas of all tributary rivers in the south half of the basin are of such characteristics and size that, although they contribute to and augment the Merrimack River flow, they are not conducive to producing damaging floods on the Merrimack River. Consequently, as the maximum discharges are produced by the headwater tributaries in the mountainous portion of the drainage area, the maximum crest of the flood at Lawrence occurs about 55 hours, or more than two days, following the flood producing rainfall. This characteristic of the river hydraulics is noted because of its importance in the selection of pumping station capacities. It was concluded that the maximum flood of record, reduced by the comprehensive reservoir program, would provide a safe design criteria for local protection works in this area. This is the same design criteria for the completed local protection projects on the Merrimack River. The maximum river stages of record, both uncontrolled and as reduced by the proposed comprehensive reservoir system are shown on Plates 9 and 13. At North Andover and south Lawrence projects, the selected grade for the top of the dikes and walls is Elevation 42.0 feet which provides a freeboard of 3 feet above the maximum stage of the 1936 flood as reduced by the reservoirs. At the west Lawrence project, the top of earth dike and concrete wall will be maintained at Elevation 55.0 which is the grade previously established for the W.P.A. project. This grade provides 4 feet of freeboard above the maximum stage of the 1936 flood as reduced by the reservoirs. At North Andover and south Lawrence there will be no velocity of flow since the high stages in these areas are the result of backwater from the Merrimack River, and the wave action will be negligible because the fetch is comparatively small. Additional freeboard is required at west Lawrence because the project borders the main river channel and comparatively high velocities of flow are anticipated. It is believed that these freeboard provisions

are adequate allowance for (1) wave action, (2) degree of accuracy in reach routing from the headwater tributaries to Lawrence and (3) effective protection against design flood until such time as the comprehensive reservoir system is completed.

5. Hydrology.- (See Appendix "A" for complete hydrological analysis.)

a. North Andover.- A 24-inch drain discharging into the Shawsheen River and a 24-inch trunk sewer that extends to the Merrimack River are located in the area affected by the proposed flood wall. (See Plate 5.) The 24-inch drain with a discharge capacity of 20 c.f.s. serves a drainage area of 65 acres. An area of 13 acres between Massachusetts Avenue and the flood wall on the west will be provided with new drains and catchbasins. The 24-inch sanitary trunk sewer, with a capacity of 6 c.f.s. serves a comparatively small area at present, but was installed for the purpose of serving 1.5 square miles of North Andover, if the development of the town warranted its extension. It is proposed to construct a pumping station east of Massachusetts Avenue as an integral part of the concrete flood wall to pump the sanitary sewage and storm run-off into the Shawsheen River during high river stages. Results of hydrologic studies of past records indicate that it is very improbable that heavy rainfall and flood stages will occur concurrently, and it is proposed to provide pumps with a total capacity of only 36 c.f.s. which is the capacity of the existing and proposed drainage systems plus the maximum estimated sewage flow in the sewerage system. Storm run-off that exceeds the capacity of the drainage system will be discharged through gated emergency outlets through the flood wall during mornal river stages, or will be temporarily ponded in the low dump area back of the flood wall during high river stages.

b. South Lawrence.- A 54-inch diameter brick sewer flowing north in Osgood Street, serves as a trunk outlet for both sanitary sewage and storm drainage in Areas "C" and "D" and for sanitary sewage from Area "E" (Plate 8.). The drainage areas are as follows:

Area "C"	- 40 acres
Area "D"	- 160 acres
Area "E"	- 104 acres
Total Area	- 304 acres

Area "C" is the low area inclosed by the proposed dike and is the approximate limit of the section inundated during the past floods. Areas "D" and "E" are at higher elevations and are not subject to flooding except during extreme high floods. The discharge capacities of the existing sewers and drains in Area "C" are approximately 28 c.f.s., compared with a computed discharge of 48 c.f.s. based on a 30 minute rainfall for a ten-year frequency. Due to the limited capacity of the existing sewers, a new drainage interceptor with gated emergency outlets through the concrete wall to the Shawsheen River will be provided

to prevent ponding in the low area in the Southern portion of the protected area during normal river stages. The capacity of the 54-inch trunk sewer is approximately 100 c.f.s. On the basis of frequency studies of Merrimack River flows, (Plate 10), it is expected that pumping will be required once every 12 years in Area "C", but in Areas "D" and "E", it is anticipated that pumping will be required only once in 110 years. New pipes and control facilities will be provided to take advantage of the difference in ground elevation and the probability of pumping requirements by separating the flow in Area "C" from "D" and "E". It is proposed to provide pumps with a total capacity of 35 c.f.s. The maximum design pumping head will be approximately 25 feet.

c. West Lawrence.- A hydrological analysis is not required for west Lawrence. The area is served by a combination sewerage and drainage system that discharges into the Merrimack River downstream from the Essex Dam, and consequently, is unaffected by backwater. During the initiation of the project, the City of Lawrence installed new drains in the area inside and adjacent to the dike to prevent local ponding. The construction of the dike and wall does not effect the present sewerage and drainage systems, and consequently, interceptors and a pumping station are not necessary. A toe drain will be included to intercept subsurface seepage during flood stages, with the flow discharging into the combination sewer system.

6. Engineering Design.-

a. Surveys.- A topographic survey was made of the North Andover flood area which included the immediate northeast bank of the Shawsheen River, its primary flood plain or "bottom land" and the northeast valley wall, together with land immediately adjacent to the top of the bank. Topographic data for the south and west Lawrence flood area sites were obtained from plans based on surveys made by the City of Lawrence in connection with the proposed W.P.A. projects for flood protection for these areas which were prepared several years ago. Additional information concerning the west Lawrence dike was obtained by a topographical survey to determine the scope of the work required to extend and complete the project initiated by the W.P.A. in 1940. These data are considered satisfactory for preliminary planning of the structures now proposed. Considerable use was made also of aerial photographs for location of topographical features and buildings in all areas.

b. Foundation Exploration.-

(1) North Andover and South Lawrence.- The foundation areas for the proposed dikes and retaining walls were explored by three (3) drill holes and thirty-eight (38) auger borings. Overburden exposures were inspected and local residents questioned regarding soil conditions. The location of the explorations are shown

on Plate 5 and the logs of borings taken at North Andover and south Lawrence are included on Plate 7. In general both projects are located on the top of the banks forming the valley walls of the Shawsheen River. The banks are covered over large areas by a surficial deposit of rubbish from one foot to several feet in depth. There are three large rubbish and ash dumps and the extent of these is shown on Plate 5. In general, these dumps were constructed by end dumping from the original valley walls, thus they extend in depth to the elevation of the present river flood plain. Beneath the rubbish fills, stratified sands and silts with some gravel phases extend to depths of 30 to 50 feet below the river flood plain. Bedrock, a granite gneiss, was encountered at drill hole D1 at a depth of 27.5 feet below the river flood plain. The rubbish and ash fill is exceedingly non-uniform but very pervious, while the underlying natural deposits have coefficients of permeability as determined from remoulded samples, as follows:

Soil	Range of Coef. Of Permeability cm. per sec.
Clean Sands and Gravelly Sands	10^{-3} to 10^{-4}
Silts	10^{-5} to 10^{-6}

(2) West Lawrence.- The foundation areas for the proposed dikes and retaining wall were explored by five (5) drill holes and six (6) test trenches. Overburden exposures were inspected. The dike construction carried out by the W.P.A. was observed and tested for compaction and permeability. The locations and logs of the drill hole explorations made are shown on Plate 12. In general, the existing portion of the dike constructed by the W.P.A. and the unconstructed portion of the dike to the southwest corner of O'Sullivan Park are and will be founded upon a deposit of loose sand with variable fine and coarse phases and occasional pockets of clayey silt. The fill encountered at the ground surface in drill holes D1 and 3 is the existing W.P.A. embankment. In the area of the concrete dike and the earth dike to the northeast, a deposit varying from 5' to 15' of fill was encountered in drill holes D2, D4, and D5. The fill is very variable and typical of unassorted rubbish, ash and earth fills. Beneath the fill, fine sand was encountered with variable fine and coarse phases. Bedrock at the site is believed to be at considerable depths but is outcropping on the south bank of the Merrimack River opposite the dike. The W.P.A. constructed the existing portion of the earth dike using predominately a fairly pervious silty sand and gravel obtained as stripping from a glacial gravel deposit. A small portion of the dike at the northeast end of the constructed portion was built of an impervious, well graded clayey sand and gravel (glacial till). The toe trench was constructed partly of washed gravel screenings 2" to 3" in size and partly of clean sandy gravel. The coefficients of permeability of the various soils encountered in the drill holes and used for construction are as

follows:

Material	Coefficient of Permeability cm. per sec.
Fine sand foundation of dike	8 to 24 x 10^{-4}
Silty sand and gravel embankment fill as placed by W.P.A.	6 to 40 x 10^{-4}
Well graded clayey sand and gravel fill as placed by W.P.A.	.6 x 10^{-4}

c. Description of Walls and Dikes.- (1) North Andover.-The protective structures for the North Andover area include an earth dike approximately 750' in length, and a reinforced concrete flood wall approximately 2300' in length. The earth dike will border the east line of Elmwood Street starting from existing ground elevation 42.0 and extending southwesterly approximately 350' to the end of the street where it will make a right angle and continue approximately 400' northwest to join the end of the proposed reinforced concrete flood wall. Space limitations prevent extension of the earth dike section beyond its juncture with the proposed concrete wall. The concrete wall will extend northwesterly along the top of the valley bank for approximately 1150' then angle northeast, crossing Massachusetts Avenue and extending along the top of the bank to meet the existing grade at elevation 42.0. The concrete wall is designed as a modified T section which will be of constant cross section throughout its entire length. For basis of design it is assumed that the flood stage is at the top of the wall (Elevation 42.0). The earth work supplementing the wall is varied to meet three different types of ground conditions. These three general types of ground conditions encountered at different places under the wall site are (1) existing ash and rubbish fill up to 14' in depth, (2) a compacted earth section built across a low area, and (3) firm existing natural ground. Where conditions (1) and (2) are encountered, the wall is to be constructed on a core of compacted impervious material extending vertically down through the pervious materials and into firm existing natural soil to a depth of at least 3 feet. Throughout the entire length of the concrete wall and of the earth dike, provision has been made for drain lines to receive and carry off surface drainage and seepage. A removable bulkhead to close the Massachusetts Avenue opening in times of flood, and facilities for storing the bulkhead when not in use will be provided. Plans and typical sections of the reinforced concrete flood wall and of the earth dike are shown on Plates Nos. 5 and 6. Channel changes in the existing channel of the Shawsheen River are contemplated to eliminate the possibility of riparian erosion which might undermine the proposed flood wall. This work will involve the construction of approximately 800 lineal feet of new channel including the excavation of approximately 11,000 cubic yards of earth from this new channel and placing it to fill the abandoned channel.

(2) South Lawrence.- The proposed system of protective structures in the south Lawrence area includes an earth dike approximately 570 feet in length and a reinforced concrete flood wall of varying cross section, approximately 3250 feet in length. The earth dike will extend from existing ground elevation 42.0 near Shawsheen Road northeasterly along the top of the valley bank to Loring Street where a concrete retaining wall parallel to the street line will retain the earth section. A removable bulkhead to close the Loring Street opening during periods of flood, and facilities for storing the bulkhead when not in use will be provided. A reinforced concrete flood wall, starting at the north line of Loring Street, will extend northeasterly along the top of the valley bank and the east side of Shawsheen Road to the south right-of-way line of the Boston and Maine Railroad and then extends westerly along the north line of Portland Street ending near Osgood Street. The wall will be reinforced concrete designed in the form of a modified T section which will vary in section with the height of the wall required to make the uniform top elevation 42.0. It is planned to install drain lines to carry off surface drainage and seepage throughout the length of the wall and dike, excepting the Portland Street section. The presence of an existing street drain along Portland Street makes the construction of additional drainage facilities unnecessary.

(3) West Lawrence.- The W.P.A. project initiated in 1940 planned for 2300 feet of earth dike and about 725 feet of concrete walls. Approximately 1000 feet of the dike were finished according to the plans. The remaining 1300 feet is in various stages of completion. Work was not started on the proposed section of the concrete wall. For the partially completed 1300 feet section of earth dike, it will be necessary to provide riprap for the protection of the river side of the dike; to install an extension of a toe drain on the land side for intercepting seepage flow; and to place topsoil and seed the dike. Some additional fill and regrading will be required for sections of the dike where the impervious fill had been placed but not shaped into the final cross section. A concrete wall will be constructed along the river bank at the O'Sullivan Park in order not to restrict the use of the Park and also for the more favorable appearance of the wall compared with the earth dike. The wall will be a modified T section similar to the proposed design for North Andover and south Lawrence. The dumped and fill material will be excavated in the wall foundation area and replaced with a compacted impervious material. The test borings indicate that the depth of this fill material averages between 8 to 12 feet. A seepage trench consisting of an open joint vitrified tile drain in a trench backfilled with a pervious gravel will be provided for intercepting subsurface flows. East of the Park a shallow dike will be constructed similar to the W.P.A. dike. A minimum of 5 feet of fill will be removed and backfilled with impervious material. A waste pile of discarded city-owned cobblestones will be available for riprapping purposes. Similar use was made by the W.P.A. in the completed portion of the earth dike, and except for small areas that have been damaged wantonly, the cobble-

stone riprap appears to be satisfactory. It is proposed to repair the existing riprap in various sections that have been damaged.

d. Drainage and Pumping.-

(1) North Andover.- The existing drains and sewers, and proposed new drains, will be piped to a central pumping station, as shown on Plate 5. The existing 24-inch drainage outlet discharging into the Shawsheen River at the Massachusetts Avenue bridge will be plugged and abandoned. All storm drainage will be diverted into a new 36-inch drain and, normally, this flow will pass through the pumping station and discharge by gravity through a 42-inch outlet into the Shawsheen River. The site of the pumping station is selected to take advantage of the line of the existing outfall sewer and the necessity of combining the existing drainage system with the proposed drains to provide a single outlet. The sewage and drainage flows will be diverted into the suction conduit of the pumping station during flood stages, and pumped into the 42-inch outlet to the river. A preliminary layout of the proposed pumping station is shown on Plate 6A. Consideration and study have been given to the characteristics and cost of various types of pumps in order to satisfy the pumping requirements outlined in Paragraph 1 (g) of Appendix "A". It is desirable to have pumps with a wide range of operating flexibility to meet the anticipated variable inflows from sewage and drainage which may range from 1 to 36 c.f.s. As a result of this study, it is considered essential to provide pumps of the following capacities and power units:

1 - 6 c.f.s. non-clogging centrifugal pump with electric motor drive.

2 -15 c.f.s. fixed blade propeller pumps with gasoline engine drives.

The preliminary investigations conducted by this office indicate that the available source of electric power is unreliable and cannot be maintained during critical flood periods. The utility sub-station, which serves Lawrence and North Andover, is located on the bank of the Merrimack River in Lawrence. It was flooded in the 1936 flood and was nearly inundated again during the 1938 flood. The operating room floor of the station is at approximately the same elevation as the stage of the 1936 flood as reduced by the comprehensive reservoir system. It is, therefore, proposed to use gasoline engine drives for the large propeller pumps. The non-clogging centrifugal pump whose prime function is to pump sewage and a small runoff against low river stages will be equipped with a float valve controlled electric motor drive. An 18-inch return line, controlled by a gate valve operated from the pump room floor, will be installed to obtain flexibility of pump operation and to enable the testing of pumps by recirculation of the discharge. Additional range in discharge may be obtained by operating the gasoline engines at variable speeds. The use of the return line and operation at variable speeds will, undoubtedly, sacrifice some of the pump's

efficiency, but, because of the infrequent use of the pumps, it is believed that flexibility of operation to satisfy the many possible conditions of sewage and storm run-off is a more essential criteria than pump efficiency and economy of operation. Consideration has been given to the possibility of using adjustable blade propeller pumps to obtain the desired range in pumping capacity, but it was concluded that this type of pump was not warranted due to the greater cost, and the additional mechanism to be maintained in operating condition by the municipality.

(2) South Lawrence.- (a) The existing and proposed sewerage and drainage system for Areas "C" and "D" is shown on Plate 5. Sewerage in the existing combination sewers in Salem, Greenfield, Market and Portland Streets, Area "C", flows westerly to the 54-inch trunk sewer on Osgood Street. The proposed system, consisting of (1) existing combination sewers; (2) relief outlets through the concrete flood wall in the low section on the south side of Area "C"; (3) interceptor sewers on Loring and Osgood Streets, connecting the existing sewers on Salem, Greenfield, Market and Portland Streets, and the proposed drains and sewers along the flood walls and dikes with the proposed pumping station; and (4) a control chamber on Osgood Street to prevent backwater in Area "D"; is designed to meet the following requirements:

1. Eliminate ponding of run-off in Area "C" resulting from the construction of the concrete wall. The capacity of the proposed system is not large enough to handle run-off from heavy rainfall usually occurring prior to flood stages. The run-off which would normally flow overland to the Shawsheen River will be collected by the drainage interceptor adjacent to the concrete wall and be discharged through the relief outlets. Flap gates will be installed on the relief outlets to prevent backwater from entering the protected area during flood stages.

2. Discharge the surface run-off and seepage intercepted by the flood walls and dikes in Area "C". During river stages below El. 29.0' discharges will not be admitted to the pumping station but will flow directly to the 54" trunk sewer. During river stages above El. 29.0', flow will be admitted to the pumping station and discharged to the control chamber on the trunk sewer. Gate valves will be provided to prevent backwater from entering the existing sewers on Salem and Greenfield Streets and to reverse the flow in these sewers to the interceptor on Loring Street.

3. Diversion of the sewage and nominal drainage discharge from Areas "D" and "E", to the pumping station for river stages above El. 34.0'. When protection is required for Area "C" during flood stages, a gate will be closed to prevent discharge into

the 54-inch sewer, and the flow will be diverted to the pumping station. The pumping station discharge will be carried in a 42-inch conduit to control chamber on the 54-inch trunk main. When protection is necessary for both Areas "C" and "D" during higher flood stages, a gate located on the 54-inch main in the control chamber will be closed, the gate at the entrance of the pumping station will be opened and the discharge from all inflow pipes will be diverted to the pumping station. The discharge from the pumping station will be carried through the 42-inch discharge conduit back to the 54-inch main. Computations show that with all three pumps operating during high river stages, there is a drop of one foot in the hydraulic gradient due to frictional losses from the pumping station to the Merrimack River. If the flood stage reached elevation 40.0 (2 feet below top of the wall) water level at the pumping station would be about elevation 41.0. Due to the drop in gradient, it will be necessary to reconstruct the top of three existing manholes to make them watertight under the head created by backwater and pumping. A preliminary layout of the proposed control chamber and pumping station is shown on Plate 6A. The site selected for the pumping station is a vacant lot located advantageously to the existing and proposed sewers.

(b) It is proposed to provide pumps of the following capacities and power units for the Lawrence pumping station:

1 - 5 c.f.s. non-clogging centrifugal pump with electric motor drive.
2 - 15 c.f.s. fixed blade propeller pumps with gasoline engine drives.

The selection of gasoline engine driven propeller pumps and an electric motor driven centrifugal pump for south Lawrence is based on the same considerations that governed the selection of pumps for the North Andover project. Sufficient flexibility of operation can be obtained by the installation of an 18-inch return line controlled by a gate valve and by operation of the gasoline engines at variable speeds. The proposed pumps will have a total capacity of approximately one-third (1/3) the discharge capacity of the combination 54-inch trunk sewer. The installation of pumps and stand-by units to provide for the capacity of the trunk sewer would increase the cost of the Lawrence project by approximately $50,000. It is considered that the benefits obtained by this additional protection do not justify the insurance since the chances are extremely remote that a storm of high intensity will occur simultaneously with high flood stages. In terms of probability (Plate 10), it would require a storm with a magnitude to be expected once in one (1) year occurring concurrently with a flood stage to be expected once in one hundred (100) years. If a storm which produced run-off in excess of the pumping capacity did take place during

flood stages, the result would be surface storage in low areas and a few flooded cellars in Areas "D" from backwater in the sewers. The duration of these conditions would be short without serious inconvenience to the affected areas. It is concluded that the 35 c.f.s. is the theoretical practicable capacity of the pumping units.

e. Alternate Protective Plans Considered.- Two other schemes for the protection of these areas have been considered but neither have appeared to be economical. Brief descriptions of the plans considered are outlined in the following paragraphs:

(1) Construct an earth dam across the Shawsheen River just upstream from the Boston & Maine Railroad bridge. On the aerial photograph (Plate 3) this site would be approximately a straight line extension of the dike along the side of Portland Street. Gated conduits would permit ordinary discharges to flow normally to the Merrimack River. During flood stages the gates would be closed and the flow in the Shawsheen River would be pumped against the flood stage of the Merrimack River. As it is possible that the flood flow of the Shawsheen River would synchronize with the flood crest on the Merrimack River, it would be necessary to provide pumping capacity of approximately 1500 c.f.s. The cost of the pumping station and equipment alone exceeds the entire cost of the recommended project. In addition to this large pumping plant, it would be necessary also to have pumping stations to (1) dispose of the sewage from the North Andover section, and (2) prevent backwater and flooding in the Lawrence section from the trunk sewer outlet.

(2) The second plan, to eliminate the cost of pumping the flow in the Shawsheen River, was the construction of two dams across the Shawsheen, one located as described in the preceding plan, and the second located upstream about opposite Elmwood Avenue in North Andover. These two dams would be connected by a conduit sufficient in size to carry the flood discharges of the Shawsheen River. A small pumping plant would be required to dispose of the local sewage and drainage. The cost of this plan greatly exceeds the proposed system of dikes and walls.

7. Cost Estimate.- The estimated costs of the local protection projects for flood control at North Andover and south and west Lawrence are as follows:

a. North Andover.-

Wall and dike	$185,800
Pumping Station and drains	67,500
Channel realignment	5,400
Subtotal.....	$258,700
Engineering, Inspection, Overhead and Contingencies (25%).....	64,700
Total construction costs.....	$323,400

b. South Lawrence.-

Wall and dike	$136,200
Pumping Station and drains	85,400
Subtotal.....	$221,600
Engineering, Inspection, Overhead and Contingencies (25%).....	55,400
Total construction costs.....	$277,000

c. West Lawrence.-

Wall and dike	$ 41,800
Engineering, Inspection, Overhead and Contingencies (25%).....	10,450
Total construction costs.....	$ 52,250

d. Total Estimated Cost:- (3 sections) $652,650
(For itemized analysis of cost estimates, see Appendices B, C and D).

8. Benefits of Projects.-

a. Methods of Analysis.- Reference is made to Section "B", "Data for Economic Justification", in "Appendix to Accompany Survey Report for Navigation, Flood Control and Water Power, Merrimack River, Massachusetts and New Hampshire" submitted by this office, dated 1 April 1940. Section "B" described in detail the methods used in the economic analysis of flood control reservoirs based on direct and indirect flood losses. The same method applies to the economic analysis of local protection projects.

b. Determination of Benefits.- The proposed local flood protection works will supplement the effectiveness of flood protection provided at the damage centers by the upstream reservoirs and will increase the total benefits accruing to the comprehensive flood control program. The additional benefits resulting from the local protection projects have been evaluated and totalled with the benefits secured from the balance of the comprehensive flood control program to determine the economic justification for the proposed work. The proposed dikes and walls have been found to provide the most economical means of supplemental flood protection for North Andover and Lawrence. In addition to providing protection from floods, it is believed that the proposed project will be very beneficial to the general welfare of the community by promoting development of the low areas and enhancing property values. The costs and benefits involved in the comprehensive flood control program are summarized in the following tabulation:

Summary of Benefits and Costs

(1) Construction Costs:

(a) Local protection projects		
North Andover	$323,400.	
South Lawrence	277,000.	
West Lawrence	52,250.	
Subtotal		$ 652,650.
Other projects*		660,500.
(b) Flood Control Reservoirs**.....		16987,000.
Total Construction Costs.....		$18,300,150.

(2) Annual Carrying Charges:

(a) Local protection projects		
North Andover	$ 13,880.	
South Lawrence	11,880.	
West Lawrence	2,300.	
Subtotal		$ 28,060.
Other projects*.....		35,740.
(b) Flood Control Reservoirs**		788,970.
Total Annual Carrying Charges		$ 852,770.

(3) Total Annual Benefits:

Based on comprehensive flood control program, including reservoirs and local protection.....$1,060,500.

(4) Ratio of Annual Benefits to Annual Carrying Charges: 1.24

* Includes completed local protection at Lowell, Massachusetts and proposed project at Nashua, New Hampshire.

** Includes completed reservoirs at Franklin Falls and Blackwater, and proposed reservoirs at Bennington, Beards Brook, West Peterboro, and Mountain Brook.

9. Local Cooperation.-

a. Views of Local Interests.- The proposed projects have been discussed in a joint meeting on 9 March 1944 with the officials of North Andover and Lawrence in order to determine the attitude and local

interest of the affected communities. The Lawrence projects were previously initiated and sponsored by the City as a W.P.A. project and the proposition that the flood control protection measures might be completed during a post-war construction program was favored by the Mayor and the City Council. The contemplated W.P.A. flood control projects in Lawrence in 1939 caused considerable agitation in North Andover for similar projects for equal protection. Consequently, the Board of Selectmen of North Andover indicated their interest and desire for the proposed project. Both municipalities expressed the urgency for flood protection to eliminate further repetition of flood damages that have occurred during the past ten (10) years.

b. Extent of Local Cooperation Required.- In accordance with Section 3 of the Flood Control Act, approved 22 June 1936, the local municipalities will be required to furnish all lands, easements, and rights-of-way necessary for the construction of the project. The estimated costs to local interests for these items is $10,000 for North Andover, $5000 for south Lawrence and $4000 for west Lawrence.

c. Assurance of Local Cooperation.- The City of Lawrence, by official action of the City Council and the Mayor on 13 March 1944, and the Town of North Andover in a special town meeting held 27 March 1944, have furnished the necessary assurance that the municipalities will (1) furnish without cost to the United States, all lands, easements, and rights-of-way necessary for the project; (2) hold and save the United States free from all claims for damages due to the work; and (3) maintain and operate the project without expense to the United States, in accordance with regulations prescribed by the Secretary of War. Photostat copies of the resolutions adopted by Lawrence and North Andover are attached as Appendices "F" and "G".

10. Time Required for Construction.- It is estimated that the projects proposed herein can be completed in one construction season.

11. Recommendation.- It is recommended that the local flood protection works proposed herein, at an estimated cost to the United States of $652,650 be selected as a definite project to supplement the comprehensive plan for reservoirs and related flood control works for the Merrimack River Basin as authorized by the Flood Control Acts of 1936 and 1938.

HOMER B. PETTIT
Colonel, Corps of Engineers
District Engineer

APPENDIX "A"

HYDROLOGY OF THE DRAINAGE AREAS

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APPENDIX "A"

HYDROLOGY OF THE DRAINAGE AREAS

1. North Andover.-

a. Drainage Area.- The total area that normally drains into the Shawsheen River through the section to be protected consists of 78 acres, approximately 2/3 residential development and the remainder consisting of a gravel pit and farmland. The land slopes uniformly from approximately Elevation 80 at Railroad Avenue (Plate 8) to approximately Elevation 30 along the top of the bank at the Shawsheen River. The drainage area is divided into two sub-areas "A" and "B" for the purpose of analysis. Area "A" (13 acres) is the section bordered by Massachusetts Avenue and the proposed flood wall south of the Avenue. Most of the present surface drainage in this area is overland to the Shawsheen River without any underground drainage system. It will be necessary to install a drainage system for this area as shown on Plate 5 to carry off the storm water that will be intercepted by the dike and wall. Subdrainage also will be provided. Area "B" (65 acres) is the remainder of the drainage area lying north of Massachusetts Avenue, and except for the Gravel pit and cultivated sections, the area has a drainage system of catchbasins and drains that combine and ultimately discharge under the Massachusetts Avenue bridge into the Shawsheen River. The existing drains vary in size from 8" to 24" with a total computed capacity of about 20 c.f.s. discharging under maximum surcharge conditions. Any future development of the farm and gravel pit sections in Area "B", with corresponding extensions of the drainage system, will not necessarily augment the pumping requirements because the surface run-off can be discharged directly into the Shawsheen River.

Surface run-off exceeding the capacity of the drainage system in Area "B" will accumulate in the low area on Massachusetts Avenue at the intersection of Middlesex Street, and then flow overland on Beechwood Street and through the dump area to the Shawsheen River. The proposed dike will obstruct this natural outlet for excessive storm runoff and will require outlets through the wall to prevent ponding.

b. Sewerage.- The entire section (Areas "A" and "B"), with the exception of a few isolated residences, has a separate sanitary sewerage system. A 24-inch trunk sewer along the east side of the Shawsheen River intercepts the street sewers and carries the sewage to the Merrimack River. The 24-inch trunk sewer extends south on Massachusetts Avenue only as far as Beechwood Street. If North Andover develops sufficiently, the trunk sewer will be extended west on Beechwood Street to the Shawsheen

River and then south along the east bank of the river. Approximately 1.5 square miles of North Andover would be served by this sewer if such an extension were required. The capacity of the existing 24-inch sewer is 6 c.f.s. which is adequate for the ultimate scheme, and is more than required for the present development of the town. The estimated rate of sewage flow based on the present population with reasonable assumption of growth is as follows:

Population = 20 persons per acre
Daily Discharge = 100 gallons per capita
Load Factor (rating of maximum to average flow) = 3

The average flow, based on these assumptions, is about 0.24 c.f.s. with a maximum rate of flow of about 0.72 c.f.s.

c. Storm Rainfall.- The rainfall intensity data at Boston, Mass., for various durations and frequencies were used for this study since the rainfall records for Lawrence or North Andover are not suitable for use as maximum precipitation rates for storms of comparatively short duration. The curves (Plate 11 A) were prepared by Charles W. Sherman and published in the A.S.C.E. Transaction 1931, p. 951, vol. 95. The curves for the 1 and 2 year frequencies have been revised to include the rainfall data for the years 1929 to 1944 by Fay, Spofford, and Thorndike, Engineers, Boston. The rainfall intensity was checked and found to be slightly less than the data prepared by David L. Yarnell and published in the United States Department of Agriculture Publication No. 204. The hydrological analyses of these two sites are based on rainfall rates for 15, 30 and 60 minute durations for various frequencies in years.

The time of concentration was obtained to determine the required duration of rainfall intensity to produce the maximum discharge, and was computed by assuming an average velocity of flow from the most distant part of the drainage area to the pumping station by measuring the distances along either existing or proposed drain lines. The computed time of concentration for Area "A" is 15 minutes and for Area "B" the time is 30 minutes. The 15 minute rainfall intensities were used for determining the required size of drains in Area "A" and 30 minute intensities were used for determining the theoretical pumping capacity from the total area. The maximum rainfall in inches per hour for various frequencies are tabulated below:

Frequency In Years	15-Minute Duration	30-Minute Duration
1	1.9	1.3
2	2.2	1.5
5	2.8	1.9
10	3.4	2.4
25	4.4	3.0

d. Storm Run-off.- The storm run-off was based on the Rational Formula Q = CIA, in which

Q = Run-off in c.f.s.
C = Coefficient, dependent on the soil conditions and characteristics of the topography, expressing the ratio of the rate of run-off to the rate of rainfall.
I = The maximum average intensity of rainfall over the entire drainage area, in inches per hour, which may occur during the time of concentration.
A = Drainage area in acres.

An average value of 0.50 for the coefficient "C" was selected for the residential areas because of the difficulties in obtaining accurate data on the types, slopes, and relative imperviousness of the different surfaces and the uncertainties of the future development of the area. Ten acres of Area "B" consist of open fields and a large gravel pit. The pit is excavated below the surrounding grade and collects the runoff from the surrounding area. The selected coefficient of run-off for this section is 0.10. Using these arbitrary coefficients, the value of "C" for Area "A" (13 acres) is 0.50, and for Areas "A" and "B" combined is 0.45.

The theoretical amount of storm run-off to be provided for was computed from the preceding data and is summarized as follows:

	Area "A" (15-Minute Duration)		Areas "A" & "B" (30-Minute Duration)	
Frequency in Years	Rainfall Rate in in./hr.	Storm run-off c.f.s.	Rainfall Rate in in./hr.	Storm run-off c.f.s.
1	1.9	12.4	1.3	45.6
2	2.2	14.4	1.5	56.2
5	2.8	18.2	1.9	70.2
10	3.4	22.1	2.4	84.3
25	4.4	28.6	3.0	105.5

Consideration was given to other accepted methods of computing surface run-off such as the Burkli-Ziegler Formula and the synthetic unit hydrograph, but due to the nature and size of the drainage area and the lack of basic data, it was concluded that the Rational Formula was the most applicable.

e. Seepage.- A seepage allowance of .0001 c.f.s. per linear foot per 10 feet of head is provided as a result of the subsurface explorations at the site and an analysis of the material encountered. This allowance results in a total seepage flow of about 1/4 c.f.s. at North Andover.

f. Capacity and Design of Proposed Drains.- The existing drainage system in Area "B" has a capacity of approximately 20 c.f.s. which, for 65 acres and a run-off coefficient of 0.5 is theoretically adequate for a rainfall of about 0.6 inches per hour. Excess storm run-off will result in either, or a combination of, surface storage or overland flow to the Shawsheen River through Area "A". The drainage system in Area "A" is designed to have emergency outlets with flap gates in the flood walls to prevent ponding at the walls, and with drain interceptors to discharge the normal surface run-off when the flap gates are closed by the backwater head.

The proposed drain interceptor in Area "A" varying in size from 12 to 24 inches will have a capacity of 12 c.f.s. This proposal interceptor, in conjunction with the emergency outlets to the Shawsheen River, will discharge all surface run-off from Area "A" and excess run-off from Area "B" occuring simultaneously with flood stages below elevation 33 feet (approximate low ground elevation). Above elevation 33 the emergency outlets become inoperative limiting the runoff discharge to the full capacity of the proposed interceptor which is equivalent to a rainfall intensity of a one year frequency occuring simultaneously with a flood stage of a 70 year or greater freqnency. In the event rainfall of greater intensity occurred simultaneously with flood stage at elevation 33 or above temporary localized ponding would result which would cause some inconvenience in limited areas without causing material damage. The simultaneous occurence of these rainfall and flood stage conditions is extremely remote and greatly exceeds actual conditions of record as illustrated on Plate 11.

The drain interceptor will commence at Elmwood Street and extend north-westerly along the flood wall to Commonwealth Avenue, then flow northeasterly in Commonwealth Avenue and combine with the existing 24" drain in Massachusetts Avenue.

The existing drain extends along Massachusetts Avenue and discharges into the Shawsheen River at the highway bridge. It is proposed to abandon this out-fall and to install a 36 inch drain from the point of intersection of the two 24 inch drains to the site of the pumping station and thence to the Shawsheen River.

The total capacity of the drainage system from Areas "A" and "B", including local drainage in the vicinity of the pumping station, is approximately 34 c.f.s. Drain capacities are computed on the assumption that the drains are flowing full and under a surcharge head with the maximum water surface elevation at the ground level, and using a value of "n" in Kutter's formula equal to 0.015. The layout of the proposed drainage system for Area "A"; its combination with the drainage system in Area "B"; the location of the pumping station; and the point of discharge to the Shawsheen River, are shown on Plate 5.

g. Pumping Requirements.- The total inflow capacity to the pumping station is approximately 36 c.f.s., including 2 c.f.s. for sanitary sewage. The range in the inflows that may be anticipated vary from the normal sewage discharge of less than 1 c.f.s. to this maximum flow with storm run-off of 36 c.f.s. The pumping head, produced by the river flood stages, is also variable depending on the magnitude of the flood at the time of a storm run-off producing rainfall.

A study (See Plate 11D) was made to determine the maximum river stage at North Andover versus the maximum storm run-off from Areas "A" and "B". Seasonal frequency curves were prepared as follows: (1) Maximum 1/2 hour rainfall intensity in inches per hour at Boston, Mass. based on records for the period 1905-1940 incl., (see Plate 11B) and (2) Maximum river stages at the South Lawrence and North Andover Projects based on records by the Essex Co. at Lawrence, Mass., for the period 1849-1938 incl. (see Plate 11C). River stages were not corrected for the effect of existing and proposed flood control reservoirs. The above curves were constructed independently without any synchronized relationship between maximum rainfall intensity and maximum river stage, except for seasonal correlation of each.

Seasonal run-off frequency was determined from plate 11B for Areas "A" and "B" by assuming that the frequency of rainfall intensities at North Andover were the same as for Boston, Mass. Run-off was determined by the Rational Formula

Q = CIA in which it was assumed that (1) "C" varied from 0.5 during the summer season to 0.8 during the winter season, except for the gravel pit where "C" was assumed to be equal to 0.1 for all seasons, (2) "I" is the maximum 1/2 hour (time of concentration for Areas "A" and "B") rainfall intensities obtained from Plate 11B, and (3) "A" is the area equal to 78 acres. The probable seasonal runoff in c.f.s. for North Andover is shown on Plate 11D.

An attempt has been made to also show on this same Plate 11D the relation between the storm runoff and the flood stage. This correlation combines the data summarized on Plates 11B and 11C by making the assumption that the seasonal rainfall for a given frequency occurred simultaneously with the seasonal flood stage of the same frequency. For example: the spring runoff from a 10 year frequency rainfall, (rainfall - 0.95 inch per hour, computed runoff = 46 c.f.s.) is plotted against the spring stage to be expected for a 10 year frequency. It should be emphasized that the simultaneous occurrence of these two factors will not take place in a 10 year frequency for it is quite improbable that the intense rainfall will synchronize with the peak flood stage. If there were no physical relationship between the rainfall and flood stage the mathematical chances of their occurring simultaneously for the particular point illustrated would be once in 9,000 years assuming the flood stage to have a duration of 24 hours. Undoubtedly, however, there is some meterological correlation between the storm that produced the flood, and the possibility that the storm might be of such duration to still produce high intensity rainfall concurrent with the flood stage. As the probable frequency of this occurrence is difficult to evaluate, the simplified curves on Plate 11D which show a coincident occurrence of flood stage and rainfall for a given frequency, present a very conservative illustration for the requirements of storm water pumping.

On the basis of the above study, and of the capacity of existing and proposed drains, it is proposed to install pumps with a total capacity of 36 c.f.s. at river stage 39.0 feet which is the full capacity of the drains coincident with the stage of the design flood. It is proposed to install one (1) 6 c.f.s. centrifugal pump to provide for sanitary sewage and small rainfall occurring at flood stage and two (2) 15 c.f.s. propeller pumps to provide for the more remote pumping requirements. A pump characteristic capacity curve plus the discharge through the flap gates is superimposed on Plate 11D which shows that the selected pumps are sufficient to provide ample protection. It is believed unnecessary to provide greater pumping capacity because the synchronized occurrence of severe rainfall and high flood stage is extremely remote. In the event greater rainfall intensity or the failure of one pump did occur during flood stage at elevation 34.0 or above, there would be some surface storage and a few flooded cellars, but the duration of these conditions would be short, without serious inconvenience to the area. Further discussion concerning the selection of pumps is included in paragraph 6 d. (1) of the report.

2. Lawrence.-

a. Drainage Area.- The hydrologic study of the drainage area is governed by the existing sewerage and drainage systems which necessitate considering separate sections of the area individually. A 54-inch diameter brick sewer in Osgood Street serves as a trunk outlet for both sanitary sewage and storm drainage in Areas "C" and "D", and for sanitary sewage from Area "E" (Plate 8). Descriptions of the different areas are as follows:

(1) Area "C" consists of approximately 40 acres bounded by the proposed flood wall and Osgood Street. This area comprises the major portion flooded by past floods, and is the area obtaining the direct benefit of the flood wall. Surface run-off exceeding the capacity of the drainage system will flow overland across Salem Street to the Shawsheen River. The proposed dike will obstruct the natural drainage for excessive storm run-off, and will require outlets through the wall to prevent ponding. Sewerage and drainage problems that arise are discussed in paragraph 6 d. (2) of the report and paragraphs 2 f. and 2 g. of this Appendix. The existing combination sanitary and storm sewers discharging into the 54-inch trunk main in Osgood Street are indicated on Plate 5.

(2) Area "D" consists of 160 acres and is analyzed separately because most of the area is at elevations not subject to river flooding. Pumping would be required for this area only during extremely high flood stages. Sewage and storm drainage are combined in the same system.

(3) Area "E" consists of approximately 104 acres that is above all flood levels. Only sanitary sewage discharges from this area into the trunk mains in Area "D" which eventually discharge through the 54-inch outlet to the Merrimack River. Storm run-off is intercepted by a separate system and is discharged into natural stream beds that flow into the Shawsheen River. All areas, except O'Connell Park are fully developed, as shown by the aerial photographs, Plates 2 and 3. The areas are primarily residential with some commercial establishments along the main streets in Areas "D" and "E". Surface drainage is intercepted in all the roads by a system of catchbasins and discharges into the combination sanitary and storm sewer. A large trunk main, increasing from 24 to 54 inches, located as shown on Plate 8, intercepts the lateral drains and discharges into the Merrimack River.

b. Sewerage.- Sewage flow from the entire area, Areas "C", "D" and "E", discharge into the 54-inch trunk sewer on Osgood Street. The estimated rate of sewage flow is based on the following assumptions:

Population = 30 persons per acre
Daily Discharge = 100 gallons per capita.
Load Factor (ratio of maximum to average flow) = 3.

The average flow, based on these assumptions, was computed to be 1.4 c.f.s. with a maximum rate of flow of about 4.2 c.f.s.

c. Storm Rainfall.- The rainfall data used for the North Andover anlaysis and described in paragraph 1 c of the appendix is applicable to the Lawrence project.

The time of concentration was computed to determine the required duration of rainfall intensity to produce the maximum discharge by the method described in paragraph 1d. The computed time of concentration for Area "C" is 30 minutes and for Area "D" the time is 60 minutes. Thirty (30) minute rainfall intensities were used for determining the required size of drains in Area "C" and 60 minute intensities were used for determining theoretical storm run-off from Area "C" and "D" combined. The time of concentration for Area "E" was not determined because only the sanitary sewage from that area is carried in the 54 inch trunk sewer on Osgood Street.
The maximum rainfall in inches per hour for various frequencies are tabulated below:

Frequency in Years	30-Minute Duration	60-Minute Duration
1	1.3	0.8
2	1.5	1.0
5	1.9	1.3
10	2.4	1.6
25	3.0	2.0

d. Storm Run-off.- The storm run-off was based on the Rational Formula Q = CIA used for the North Andover project and described in paragraph 1 d of this Appendix. By methods described in the referenced paragraph, coefficient "C" was assumed to have a value of 0.50 for Area "C" and a value of 0.70 for Area "D".

The theoretical amount of storm run-off to be provided for was computed from above data and is summarized as follows:

	Area "C" (30-Minute Duration		Areas "C" & "D" 1 Hour Duration	
Frequency in Years	Rainfall Rate Inches/Hr.	Storm Run-off c.f.s.	Rainfall Rate Inches/Hr.	Storm run-off c.f.s.
1	1.3	26	0.8	106
2	1.5	30	1.0	132
5	1.9	38	1.3	172
10	2.4	48	1.6	211
25	3.0	60	2.0	264

Consideration was given to other accepted methods of computing surface run-off and it was concluded that the Rational Formula was the most applicable. (See the discussion of this subject for North Andover in paragraph 1 d).

e. Seepage.- A seepage allowance of .0001 c.f.s. per linear foot per 10 feet of head is provided as a result of the subsurface explorations at the site and an analysis of the material encountered. This allowance results in a total seepage flow of about 0.5 c.f.s. at Lawrence.

f. Existing and Proposed Sanitary and Storm Sewers.- The general plan of the existing and proposed sanitary and storm sewers in Area "C" is shown on Plate 5. The existing system consists of, (1) Combination sewers along Portland, Market, Greenfield and Salem Streets, flowing westerly and discharging into the 54-inch trunk sewer on Osgood Street, and (2) storm drains from Greenfield Street and along Loring Street flowing southerly and discharging into the Shawsheen River. The City Engineer, Lawrence, has advised that no difficulty has been encountered from these combination sewers and drains, and it is concluded that the system in general can be utilized safely with proposed modifications as follows:

(1) Install new drainage and sewer interceptor varying in size from 12" to 36". This interceptor will commence along the dike south of Salem Street, intercept the two existing storm drains described above, slope to Loring Street, and then extend along Loring Street to the proposed pumping station located at the corner of Osgood Street and Market Street. The direction of flow is opposite to the surface slope and the depth of the drains, therefore, will increase from ten (10') feet at Salem Street to twenty (20') feet at Market Street. This same line will also intercept

the flow from the existing sewers in Salem, Greenfield and Market Streets, to isolate the sewage and drainage in these sewers from the flow in the 54-inch main.

(2) Install new fifteen (15") inch interceptor along Osgood Street from Portland Street to the proposed pumping station. This line will intercept the flow from the Portland Street sewer to isolate the sewage and drainage from the flow in the 54-inch main.

(3) Construct new catchbasins to intercept the run-off near the dike and connect the catchbasins to the existing sewers on Market, Greenfield and Salem Streets.

(4) Construct underdrains along the dike to intercept the seepage.

(5) Install two (2) flap-gated outlets through the dike and cross connect with the new interceptor at the low points south of Salem Street. These outlets will eliminate ponding in the low area with river stage below elevation 34 feet caused by the construction of the walls.

(6) Construct new catchbasins on Loring Street near the proposed bulkhead to intercept the run-off and connect the catchbasins to the proposed interceptor. During low river stages excess surface runoff will flow through the open bulkhead to the Shawsheen River.

(7) Install gate valves in the new manholes near 54-inch trunk line on existing sewers on Salem and Greenfield Streets.

(8) Abandon a portion of the existing sewer on Market Street between Loring and Osgood Streets.

g. Capacity of Existing and Proposed Sewers.- The total capacities of the existing sewers and drains in Area "C" during low river stages computed on the assumption that the drains are flowing full and under a surcharge head with the maximum water surfaces elevation at ground level, and a value of "n" in Kutter's formula of 0.015 are as follows:

Combination Sewer and Drains	Size	Capacity c.f.s.
Portland Street	15"	5.0
Market Street	15"	4.8
Greenfield Street	10"	3.2
Salem Street	15"	4.0
Storm Water Drains		
From Greenfield Street	15"	8.0
Loring Street	10"	3.0
Total Existing Capacity		28.0 c.f.s.

The capacities of the individual proposed sewers and drains during low river stages computed on assumptions as above are as follows:

	Size	c.f.s.
Portland Street	15"	5.0
Greenfield Street	10"	1.4
Salem Street	15"	4.6
New Interceptor on Loring Street	36"	27.0

The inflow to the proposed pumping station from Area "C" is 32 c.f.s. comprising the flow in the new interceptor and the Portland Street sewer.

In addition to the preceding proposed capacity the two gated outlets through the dike will discharge an additional 48 c.f.s. with river stage at elevation 29 feet.

The capacity of the 54 inch trunk sewer on Osgood Street based on average slopes and not surcharged is 100 c.f.s.

h. Pumping Requirements.- The pumping requirements are influenced by the frequency that pumping is required by different heights of river level, the probability of rainfall concurrent with that stage, and the ground elevations of the various areas. Curves showing the relationship between these items of consideration are illustrated on Plates 10, 11, 11a, 11b, 11c and 11e, and are described in paragraph 1 g of this appendix. Assuming that pumping is necessary when the river stage rises to an elevation approximately 6 feet below the lowest average ground level to prevent sewage from backing into cellars, pumping for Area "C" should commence with a river stage at about elevation 29.0. Similarly pumping for Area "D" should start at a river stage of approximately 34.0. Plates 11 and 11b show the rainfall intensities that have occurred simultaneously with high river stages and the probability of the seasonal

frequency of these rainfalls. These curves indicate the remote probability of high intensity rainfall occuring during flood stages, for, as noted in paragraph 4b of the report, the rainfall of high intensity usually takes place during the storm that produces the flood. Flood stages at Lawrence occur after more than two (2) days following the storm and ordinarily the center of the meteorological disturbance has passed by that time.

The pumps for the south Lawrence pumping station are selected to satisfy the criteria of the following range in operating conditions.

(1) Condition No. 1.-

(a) River stage 29.0 (frequency 12 years), rainfall negligible.

(b) Pumps operated for sewage flow from Area "C". Maximum rate estimated to be about 0.5 c.f.s.

(c) Sewage from Areas "D" and "E" will flow to the Merrimack River without any backwater damage.

(2) Condition No. 2.-

(a) River stage same as Condition No. 1, heavy rainfall.

(b) Pumps operated for both sewage flow and storm run-off from Area "C". Capacity of sewers equals 32 c.f.s. If sewer system becomes surcharged by severe storm run-off, excess water will discharge thru emergency outlets to the Shawsheen River. Some damage would result in flooded cellars, but this probability of a storm producing run-off exceeding the capacity of the sewers concurrent with high stages is very remote.

(c) Sewage and storm run-off would flow to river under existing conditions. Excess surface run-off would create ponds in O'Connell Park. It is possible that some surface run-off will flow from Area "D" into Area "C" but the flow will be intercepted by Loring Street and carried to the Shawsheen River.

(3) Condition No. 3.-

(a) River stage 34.0 to 38.8 (frequency 110 years), rainfall negligible.

(b) Pumps operated for sewage flows from both Areas "C" and "D". Maximum rate of flow estimated to be 4 c.f.s.

(4) Condition No. 4.-

(a) River stage same as Condition No. 3, nominal rainfall.

(b) If the total discharge from all areas is less than the total pumping capacity, all flow will be pumped in order to provide the maximum degree of protection to all areas.

(5) Condition No. 5.-

(a) River stage same as Condition No. 3, heavy rainfall (2 to 5 year frequency).

(b) The pumps will be operated primarily for the protection of Area "C" and if the total discharge from all areas exceeds the capacity of the pumps, then the flow from Areas "D" and "E" will be made to flow by gravity against the backwater head of the flood stage. A few basements may be flooded in Area "D", but the damage would be relatively light. The pumps would be utilized to discharge the full capacity of the existing and proposed sewage and drainage system in Area "C".

(c) If the storm run-off was greater than the capacity of the sewerage and drainage system, and the flap gates in the emergency outlets were closed by the pressure of the flood stage, then surface ponding will take place. However, the area is so flat that the depth of water will be in terms of inches, and will result in inconveniences instead of appreciable property damage.

It should be noted, that although the conditions of flood stage concurrent with the storms outlined in Conditions Nos. 4 and 5 are possible, the chances of their simultaneous occurrence is extremely improbable. Neither condition has occurred during the period of record.

To satisfy this wide range in design criteria it is proposed to provide pumps of comparable type and number as those recommended for the North Andover Project. One (1) 5 c.f.s. centrifugal pump will be provided for sanitary sewage (Conditions No. 1 and 3) and two (2) propeller pumps, each with a capacity of 15 c.f.s., will be installed for Conditions Nos. 2,4 and 5. The pumps will be capable of

discharging their rated capacity with a maximum operating head of approximately 25 feet. A pump characteristic capacity curve plus the discharge through the flap gates is superimposed on Plate 11E which shows that the selected pumps provide ample protection for Area "C" on the basis of coincident flood stages and rainfall. Further discussion concerning selection of pumps is included in paragraph 6 d (2) of the report.

APPENDIX "B"

COST ESTIMATE

LOCAL PROTECTION, NORTH ANDOVER, MASS.

Item	Quantity	Unit Cost	Cost	
Wall and Dike				
Excavation	11,000 c.y.	$ 0.60	$ 6,600.	
Stripping	4,000 c.y.	0.50	2,000.	
Backfill	2,000 c.y.	0.40	800.	
Compacted Impervious Fill	15,000 c.y.	1.25	18,800.	
Compacted Fill	11,000 c.y.	1.25	13,800.	
Sand and Gravel	5,000 c.y.	1.75	8,800.	
Topsoil	3,000 c.y.	2.75	8,300.	
Grading and Seeding	4.0 acres	375.00	1,500.	
Riprap	500 s.y.	5.00	2,500.	
Concrete	4,260 c.y.	25.00	106,500.	
Reinforcing Steel	124,000 lbs.	0.07	8,700.	
Bulkhead		L.S.	7,500.	
Cost of Wall and Dike				$185,800.
Drains and Pumping Station				
Drains		L.S.	12,500.	
Pumping Station		L.S.	55,000.	
Cost of Drains and Pumping Station				67,500.
Channel Realignment	12,000 c.y.	0.45		5,400.
Subtotal				$258,700.
Engineering, Inspection, Overhead and Contingencies (25%)				64,700.
Total Construction Costs - North Andover				$323,400.

APPENDIX "C"

COST ESTIMATE

LOCAL PROTECTION - SOUTH LAWRENCE, MASS.

Item	Quantity	Unit Cost	Cost	
Wall and Dike				
Excavation	13,700 c.y.	$ 0.60	$ 8,200.	
Stripping	600 c.y.	0.50	300.	
Backfill	3,500 c.y.	0.40	1,400.	
Compacted Impervious Fill	10,500 c.y.	1.25	13,100.	
Sand and Gravel	4,000 c.y.	1.75	7,000.	
Topsoil	2,000 c.y.	2.75	5,500.	
Grading and Seeding	2.5 acres	375.00	1,000.	
Riprap	275 s.y.	5.00	1,400.	
Concrete	3,400 c.y.	25.00	85,000.	
Reinforcing Steel	83,500 lbs.	0.07	5,800.	
Bulkhead		L.S.	7,500.	
Cost of Wall and Dike				$ 136,200.
Drains and Pumping Station				
Drains		L.S.	$ 35,400.	
Pumping Station		L.S.	50,000.	
Cost of Drains and Pumping Station				$ 85,400.
Subtotal				$ 221,600.
Engineering, Inspection, Overhead and Contingencies (25%)				55,400.
Total Construction Costs - south Lawrence				$ 277,000.

APPENDIX "D"

COST ESTIMATE

LOCAL PROTECTION - WEST LAWRENCE, MASS.

Item	Quantity	Unit Cost	Cost
Cost of Dikes and Walls:			
Excavation and Disposal	3,600 c.y.	$ 0.60	$ 2,200.
Compacted Impervious Fill	2,400 c.y.	1.25	3,000.
Compacted Random Fill	700 c.y.	0.40	300.
Pervious Fill Graded Gravel	2,000 c.y.	1.75	3,500.
Grouted Riprap	5,000 s.y.	3.00	15,000.
Concrete	425 c.y.	25.00	11,600.
Reinforcing Steel	8,000 lbs.	0.07	600.
V.C. Drain	1,300 l.f.	0.75	1,000.
Topsoil	650 c.y.	2.75	1,800.
Seeding and Grading	0.8 acre	375.00	300.
Grouting existing riprap	2,000 s.y.	1.00	2,000.
Repairs to existing riprap		L.S.	500.
Subtotal			$41,800.
Engineering, Inspection, Overhead and Contingencies (25%)			10,450.
Total Construction Costs - west Lawrence			$52,250.

APPENDIX "E"

TABLE OF COSTS AND ANNUAL CHARGES

		North Andover	South Lawrence	West Lawrence
I.	Federal Investment			
	A. Structures (50 year life) Construction costs	$297,150.	$250,750.	$ 52,250.
	B. Equipment (30 year life)	26,250.	26,250.	none
	C. Total Federal Investment	$323,400.	$277,000.	$ 52,250.
	D. Annual Federal Carrying Charges			
	1. 3% Interest	$ 9,700.	$ 8,310.	$ 1,570.
	2. Amortization, structures (0.886%)	2,630.	2,220.	470.
	3. Amortization, equipment (2.102%)	550.	550.	-
	4. Total Federal Carrying Charges	$ 12,880.	$ 11,080.	$ 2,040.
II.	Non-Federal Investment			
	A. Land and Easements	$ 10,000.	$ 5,000.	$ 4,000.
	B. Annual Carrying Charges - 4% Interest	400.	200.	160.
	C. Annual Cost of Maintenance and Operation	600.	600.	100.
	D. Total Non-Federal Carrying Charges	$ 1,000.	$ 800.	$ 260.
III.	Total Annual Carrying Charges	$ 13,880.	$ 11,880.	$ 2,300.

APPENDIX "F"

BS 800.5(Shawsheen River) -12

TOWN OF NORTH ANDOVER
COMMONWEALTH OF MASSACHUSETTS

Whereas, under the authority of Flood Control Act of 1938, approved by the President on June 28, 1938, which provides in part that "in addition to the construction of a system of flood control reservoirs, related flood control works which may be found justified by the Chief of Engineers", funds have been allotted by the Federal Government for the preparation of definite project plans for flood protection in the Town of North Andover, Commonwealth of Massachusetts;

Whereas, Section 3 of the Flood Control Act approved June 22, 1936 provides that "no money --- shall be expended on the construction of any project until -- responsible local agencies have given assurances satisfactory to the Secretary of War that they will, (a) provide without cost to the United States all lands, easements, and rights-of-way necessary for the construction of the project, --- (b) hold and save the United States free from damages due to the construction works, (c) maintain and operate all the works after completion in accordance with regulations prescribed by the Secretary of War:---":

Whereas, the proposed improvements at North Andover, Massachusetts, will be prosecuted under the direction of the Secretary of War and supervision of the Chief of Engineers, and

Whereas, the plans for the proposed improvements will be for the benefit principally of the inhabitants of North Andover, Massachusetts,

Now, therefore, be it resolved, that upon approval by the Chief of Engineers of the definite project plans and the allotment of funds for the proposed works of improvement, the Town of North Andover will, (a) furnish without cost to the United States all lands, easements, and rights-of-way necessary for said works. The lands, easements, and rights-of-way which said Town of North Andover shall furnish shall include those needed for the sites of structures, for spoil disposal areas, for access roads, and all other rights in, upon, through, or over private property which are needed by the United States in connection with the work of improvement. Maps showing the lands, easements, or rights-of-way needed for the aforesaid work will be obtained by the Town of North Andover from the United States. Detailed property surveys and title searches necessary to acquire the land or interests therein will be performed by the Town; (b) that the Selectmen of the Town of North Andover, Massachusetts, be and hereby are authorized to execute, acknowledge, and deliver, for and on behalf of the Town, to the United States any and all instruments which may be required by the United States in order to prosecute the proposed work of improvement, and to authorize and permit the said United States to act for and on behalf of the Town of North Andover, Massachusetts, under any easements, grants, or rights-of-way that have been obtained by or may be hereafter obtained by said Town of North Andover in connection with said proposed improvement.

Be it further resolved that the said Town of North Andover, Massachusetts, will hold and save the United States, its officers and employees, free from all claims for damages and from all liabilities due to the construction work and upon completion of the proposed flood control works the Town of North Andover will accept the same and will maintain and operate them without expense to the United States in accordance with regulations prescribed by the Secretary of War.

In Town Meeting: March 27, 1944

Attest: [signature]
Town Clerk

APPENDIX "G"

CITY OF LAWRENCE

IN CITY COUNCIL

March 13, 1944

33 800.5(Shawsheen River)-10

ORDERED THAT:

WHEREAS, projects have been proposed for the construction of flood control dikes and retaining walls to control the flow of the Shawsheen River within the City of Lawrence, Massachusetts and for the completion of the dike on the northerly bank of the Merrimack River east of the Municipal Pumping Station in the City of Lawrence, Massachusetts as post-war flood control projects, and

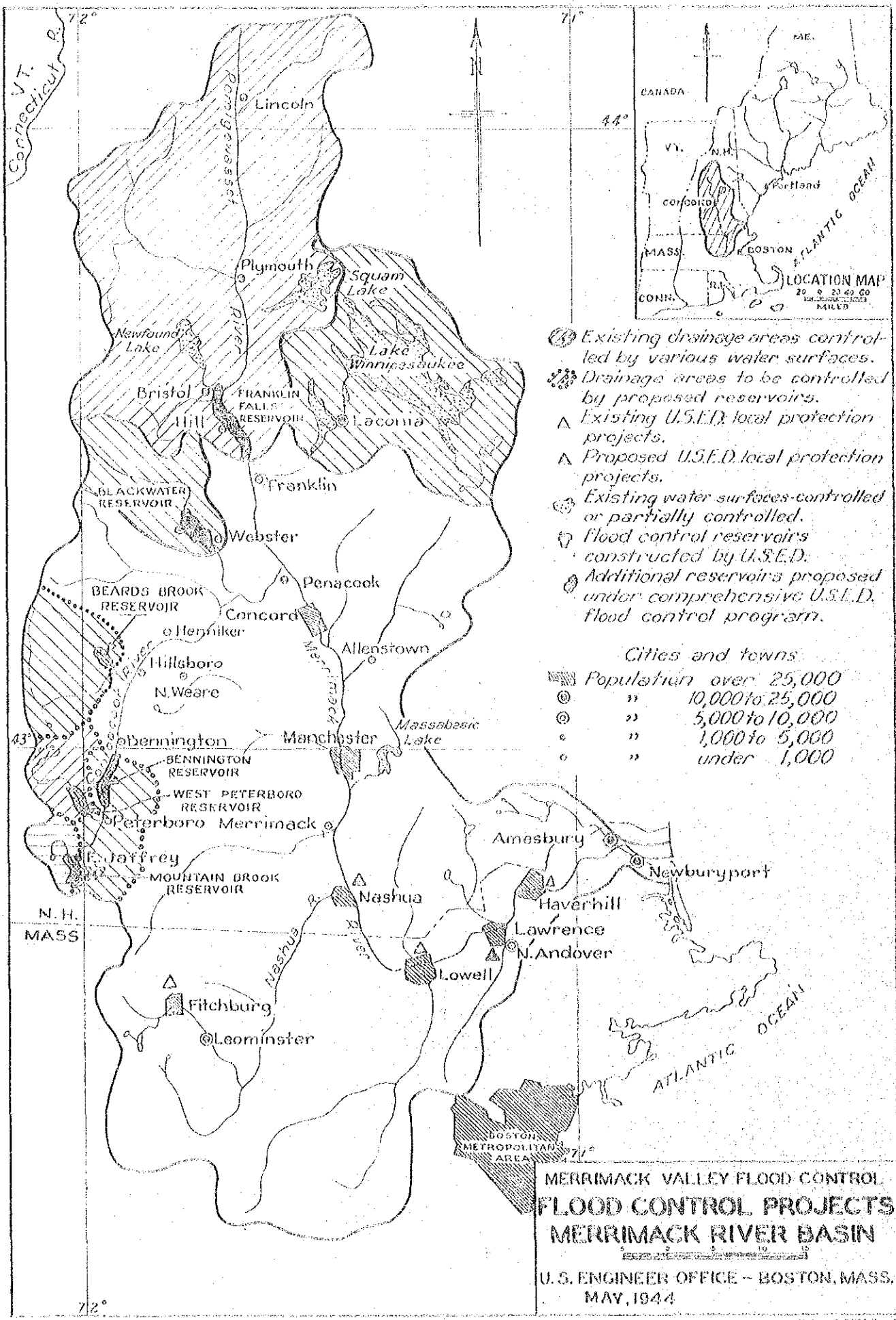
WHEREAS, the aforesaid proposed flood control projects will be constructed by the government of the United States and will benefit principally the inhabitants of the City of Lawrence, Massachusetts,

NOW, THEREFORE, BE IT RESOLVED: that upon approval of the definite project plans and the allotment of funds for the proposed flood control works by the government of the United States, the City of Lawrence will, (a) furnish without cost to the United States all lands, easements and rights of way necessary for the construction of said works. Maps showing the lands, easements or rights of ways needed for the aforesaid projects will be obtained by the City of Lawrence from the United States. Detailed property surveys and title searches necessary to acquire the land or interests therein will be performed by the City; (b) that His Honor, the Mayor of the City of Lawrence, be and he is hereby, authorized to execute, acknowledge, and deliver, for and on behalf of said City, to the United States Government any and all instruments which may be required by the United States Government in order to prosecute the proposed projects and to authorize and permit the said Government of the United States to act for and on behalf of said City of Lawrence, Massachusetts, under any easements, grants, or rights of ways that have been obtained or may hereafter be obtained by said City of Lawrence in connection with said proposed projects, and

BE IT FURTHER RESOLVED: that the said City of Lawrence, Massachusetts will hold and save harmless the Government of the United States, its officers and employees, from any and all claims for damages and from all liabilities due to the construction work upon said projects and upon completion of the proposed flood control works the said City of Lawrence will accept the same and will maintain and operate them without expense to the United States in accordance with regulations prescribed by the Secretary of War.

APPENDIX "H"

INDEX TO PLATES

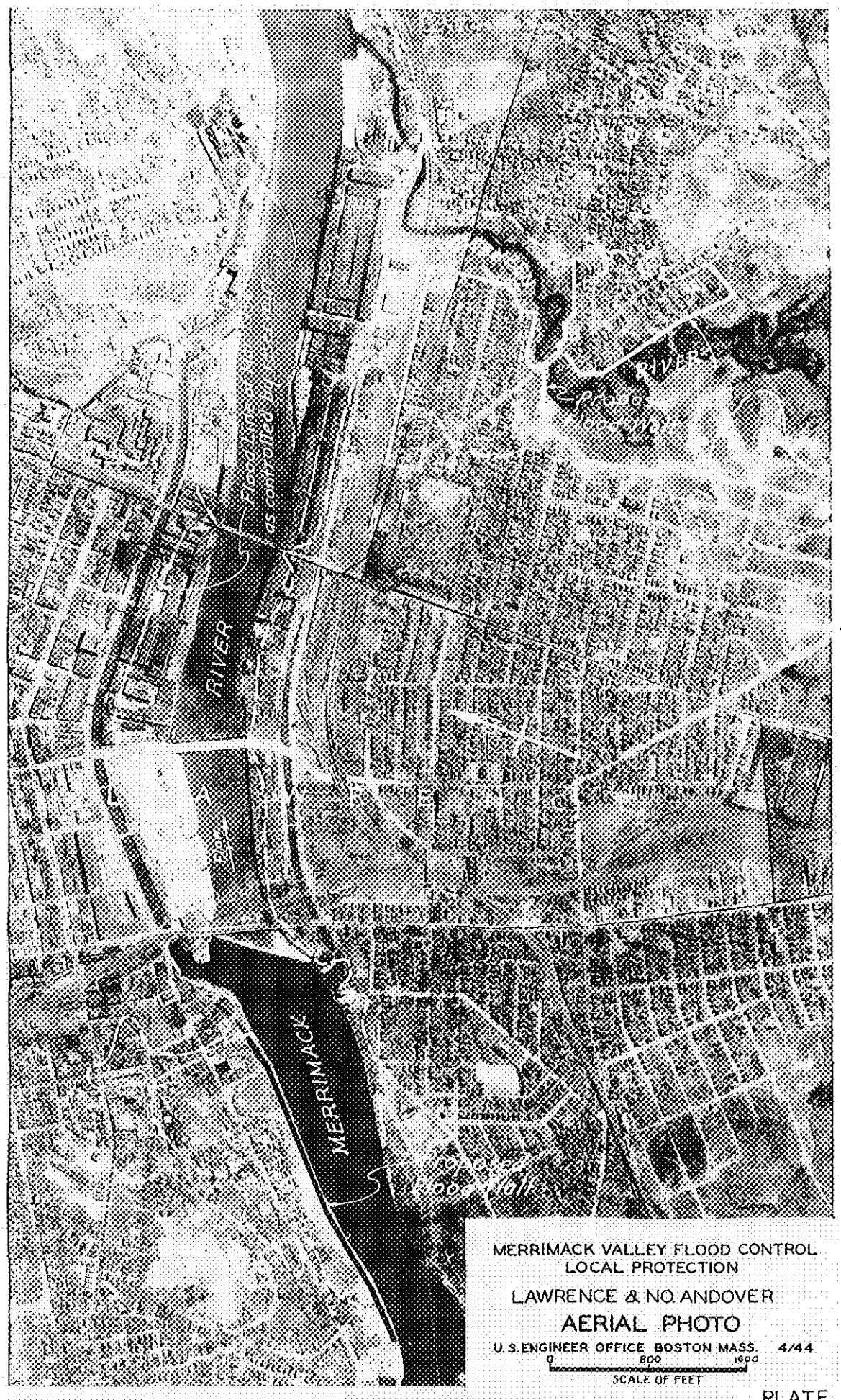


MERRIMACK VALLEY FLOOD CONTROL
FLOOD CONTROL PROJECTS
MERRIMACK RIVER BASIN

5 0 5 10 15

U. S. ENGINEER OFFICE – BOSTON, MASS.
MAY, 1944

PLATE 1



MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION

LAWRENCE & NO. ANDOVER

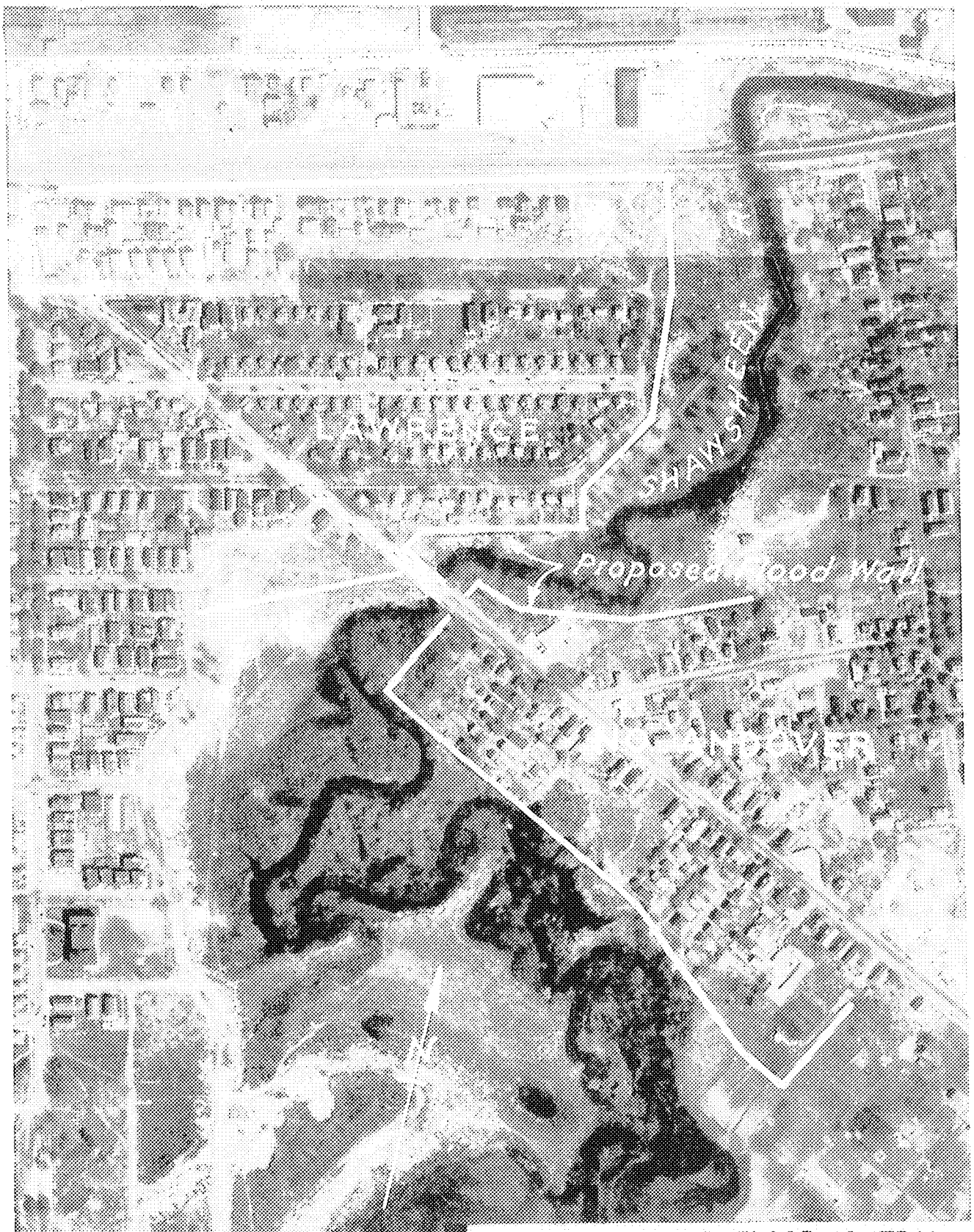
AERIAL PHOTO

U.S. ENGINEER OFFICE BOSTON MASS. 4/44

0 800 1600

SCALE OF FEET

PLATE 2



MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION
LAWRENCE & NO. ANDOVER

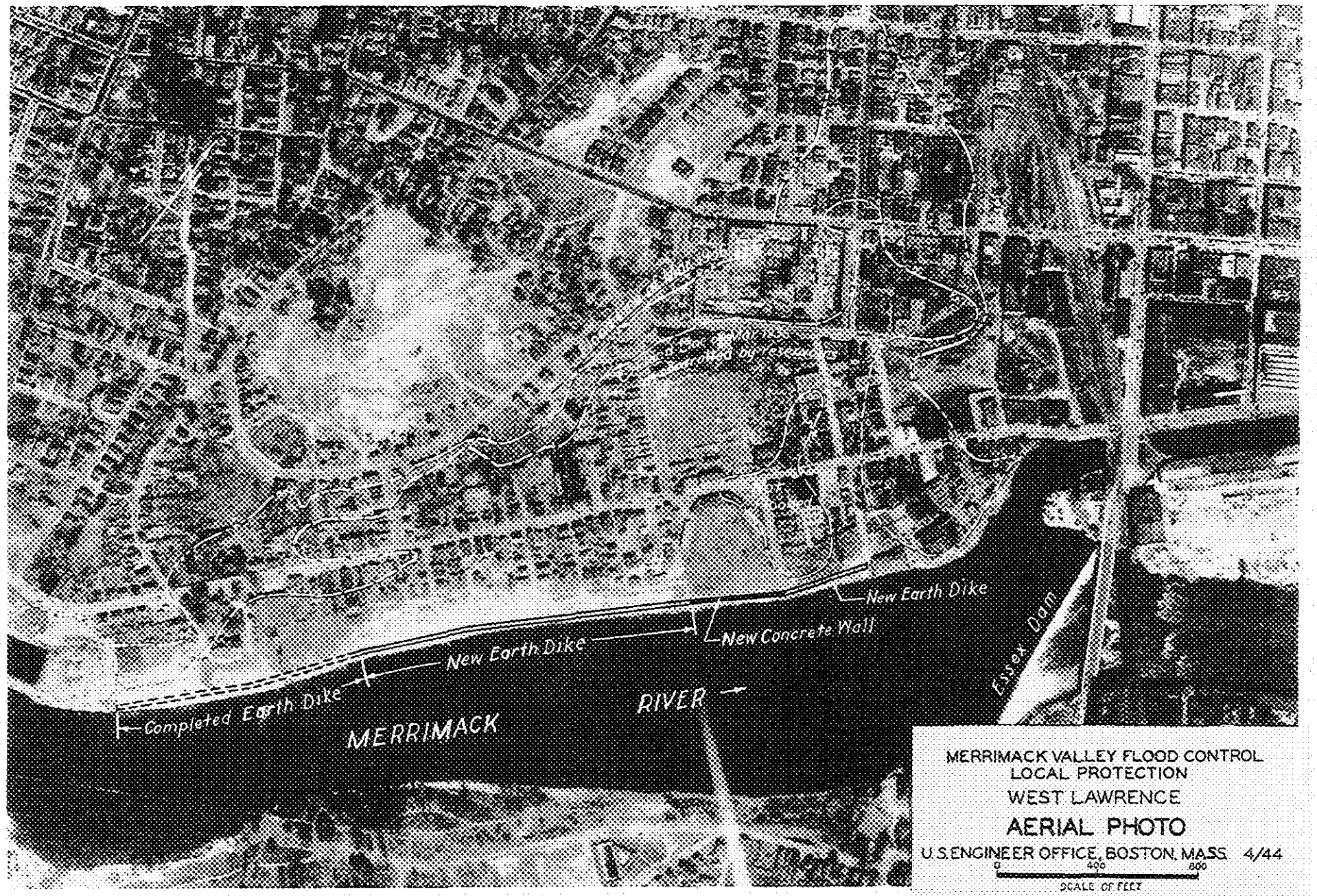
AERIAL PHOTO

U. S. ENGINEER OFFICE BOSTON, MASS., 1/44

0 400 800

SCALE OF FEET

PLATE 3



MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION

WEST LAWRENCE

AERIAL PHOTO

U.S. ENGINEER OFFICE, BOSTON, MASS. 4/44

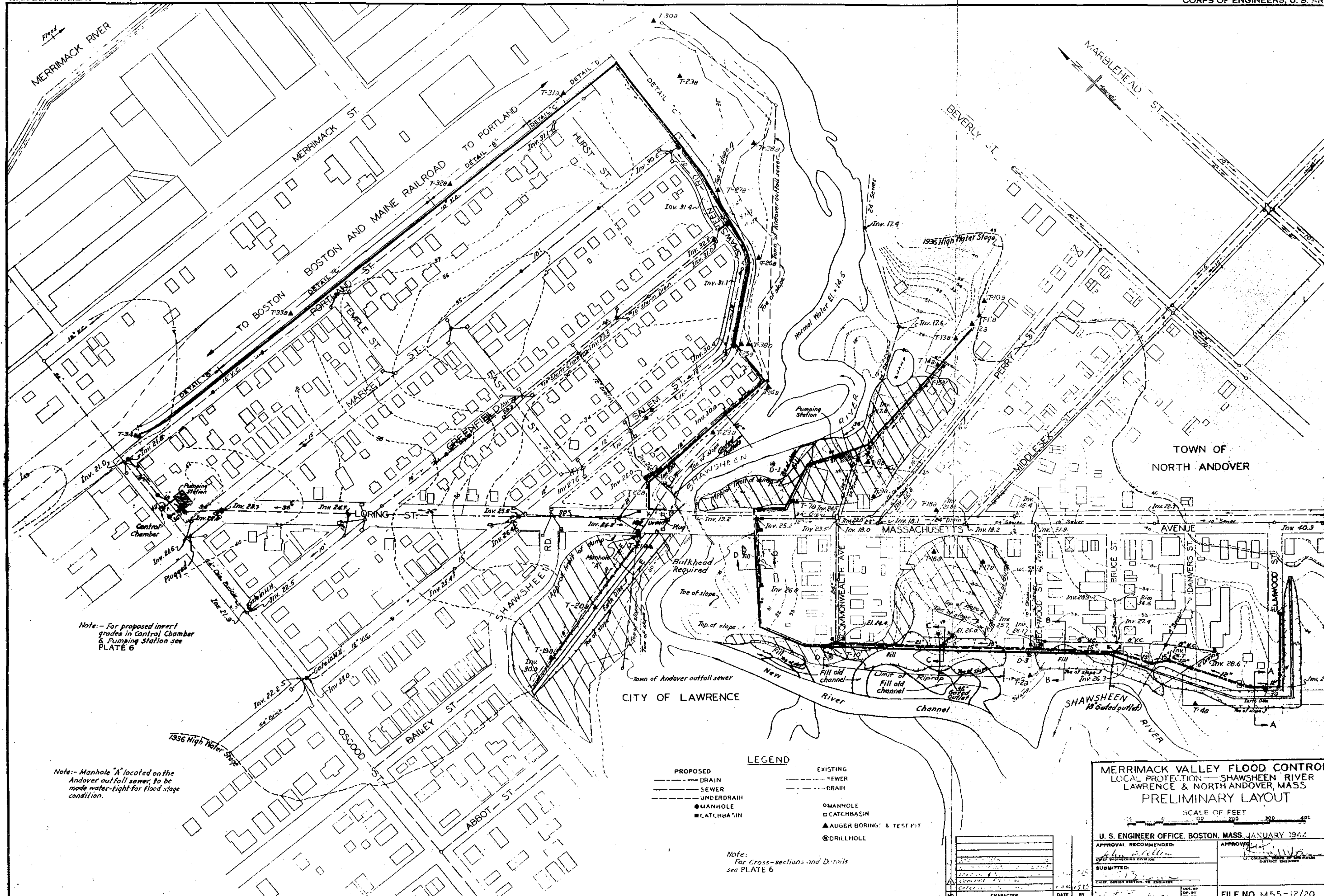
0 400 800

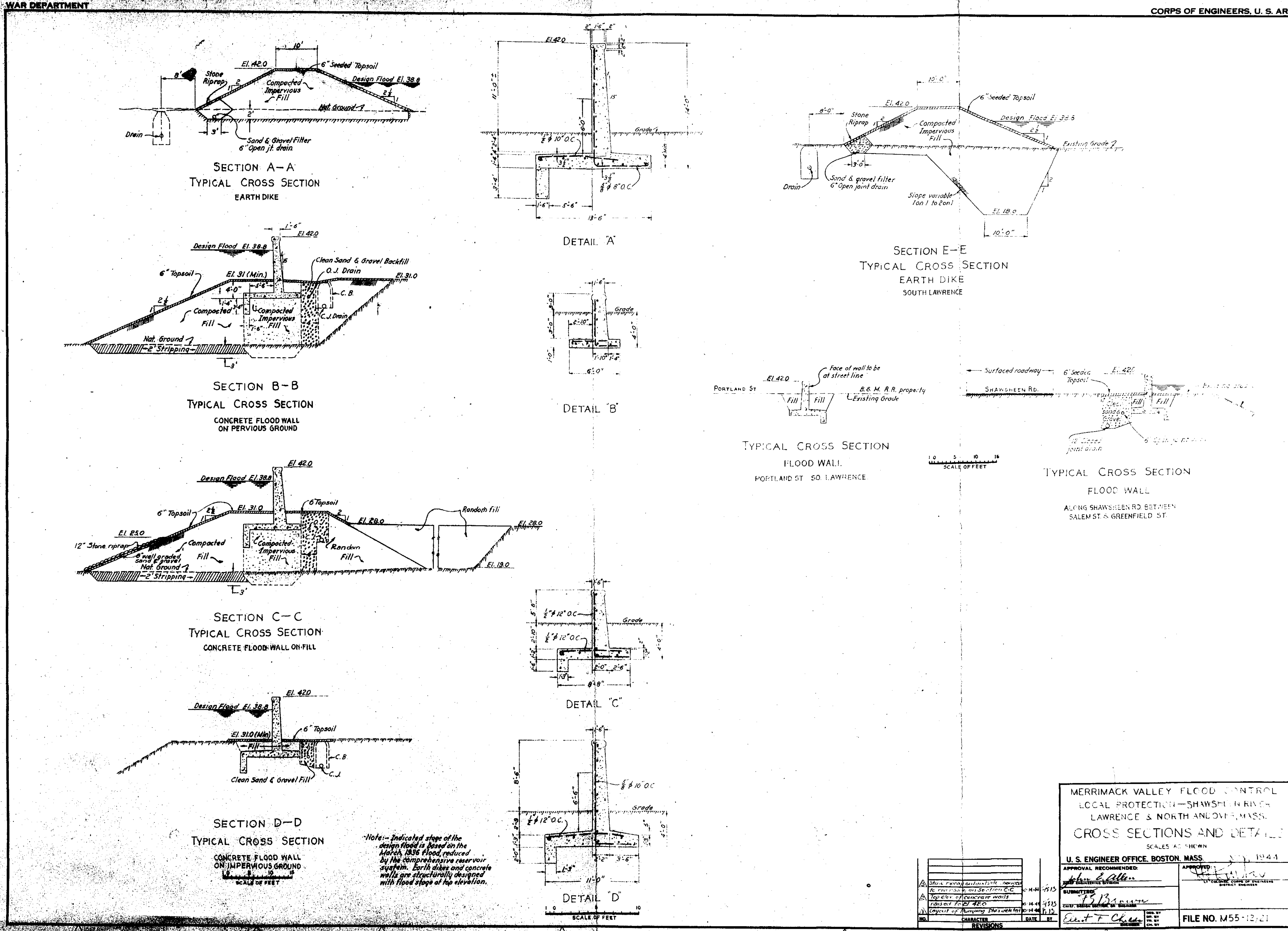
SCALE OF FEET

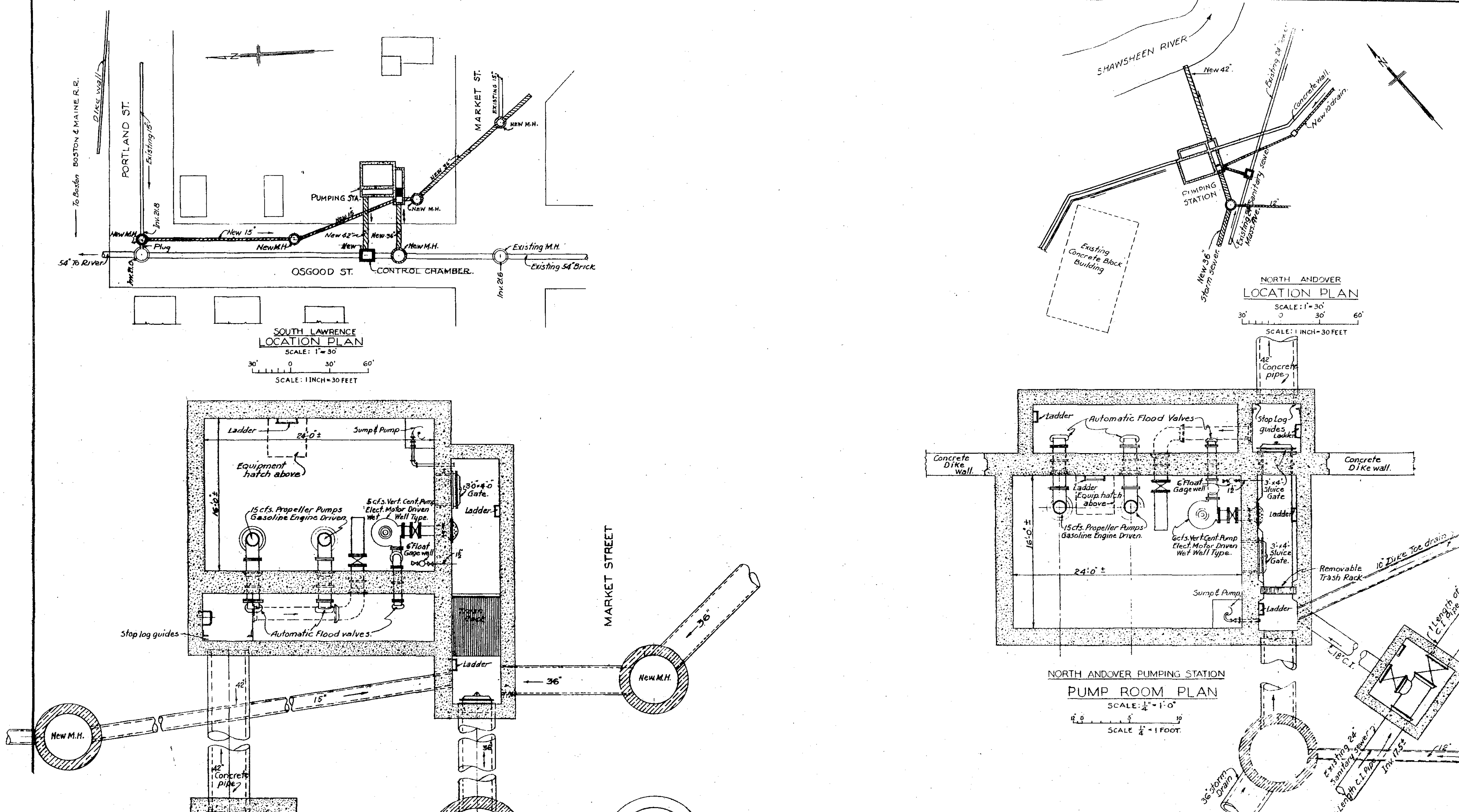
PLATE 4

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY




WAR DEPARTMENT
CORPS OF ENGINEERS, U. S. ARMY
SECTION A—A
TYPICAL CROSS SECTION
EARTH DIKE
SECTION B—B
TYPICAL CROSS SECTION
CONCRETE FLOOD WALL ON PERVIOUS GROUND
SECTION C—C
TYPICAL CROSS SECTION
CONCRETE FLOOD WALL ON FILL
SECTION D—D
TYPICAL CROSS SECTION
CONCRETE FLOOD WALL ON IMPERVIOUS GROUND
DETAIL "A"
DETAIL "B"
DETAIL "C"
DETAIL "D"
SECTION E—E
TYPICAL CROSS SECTION
EARTH DIKE
SOUTH LAWRENCE
TYPICAL CROSS SECTION
FLOOD WALL
PORTLAND ST. SO. LAWRENCE
TYPICAL CROSS SECTION
FLOOD WALL
ALONG SHAWSHEEN RD. BETWEEN SALEM ST. & GREENFIELD ST.
Note:- Indicated stage of the design flood is based on the March, 1936 flood, reduced by the comprehensive reservoir system. Earth dikes and concrete walls are structurally designed with flood stage at top elevation.
SCALE OF FEET
MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION—SHAWSHEEN RIVER
LAWRENCE & NORTH ANDOVER, MASS.
CROSS SECTIONS AND DETAILS
SCALES AS SHOWN
U. S. ENGINEER OFFICE, BOSTON, MASS.
1944
APPROVAL RECOMMENDED:
SUBMITTED:
APPROVED:
REVISIONS
FILE NO. M55-13/21
PLATE 5



MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION - SHAWSHEEN RIVER
LAWRENCE & NORTH ANDOVER, MASS.

LAYOUT OF PUMPING STATIONS

SCALES AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. JAN. 1945

APPROVAL RECOMMENDED: John E. Allen, Chief Engineering Division

APPROVED: Colonel, Corps of Engineers, District Engineer

SUBMITTED: Chief Design Section, Eng. Division

Elliot F. Chu..., Engineer

NO.	CHARACTER	DATE	BY
	REVISIONS		

FILE NO. M55-12/25

PLATE 6-A

WAR DEPARTMENT — CORPS OF ENGINEERS, U. S. ARMY

D-1 EL. 182

0.0 Black fine SAND with occasional strata of
5.8 black organic silt.
Grayish brown to brown gravelly to slightly gravelly silty fine SAND.
20.0 Gray, gravelly, silty
24.0 coarse SAND.
25.5 Brown SAND & GRAVEL.
27.5 Decomposed ROCK. Highly fractured light colored
37.5 Granite Gneiss.

D-2 EL. 230

0.0
2.2 Artificial fill.
5.5 Organic silty fine SAND
Gray, clean, fine.
8.0 silty SAND.
11.0 Grey clean fine SAND.
Brown slightly gravelly silty medium to coarse
12.0 SAND:
Light brown to gray
21.0 fine silty SAND.
Gray SILT with trace of fine sand.
36.0
Slightly gravelly light brown fine sand.
39.0 Refusal.

D-3 EL. 314

0.0 Organic topsoil.
1.0 Light gray to brown
6.0 fine to medium SAND.
Light gray to dark brown fine to medium SAND with trace of
17.0 gravel.
19.0 Light gray uniform SILT.
Light gray fine SILT and fine SAND.
28.5
Light brown fine SAND.
42.5
Light brown slightly gravelly silty fine to medium Sand.
54.5

T-1p EL. 300

0.0
Topsoil and fill
5.0
Fine silty sand — Brown loose stratified becoming wet at 12'.
12.0

T-2a

0.0 Sandy organic silt with 1.0' of topsoil.
3.4 Ground water at 1.0'
Fine gray uniform Sand
Refusal on clean
5.0 pervious gravelly sand.

T-3a

0.0 Sand and gravel fill, small amount of
0.83 organic material.
Organic topsoil, black
3.0 changing to brown, sandy
Fine, gray sand very uniform. Ground water
4.5 at 4.5'.
Fine gray sand with thin strata of plastic at 4.5'
7.8 quite compact.
Clean, coarse sand and fine gravel, very pervious. Material too coarse for
8.0 further penetration.

T-4a

0.0 Sandy brown topsoil.
1.0 Uniform fine to medium brown sand, loose with occasional gravel
Fairly compact brown sand and gravel with
5.25 gravel.
Medium to coarse sand with some gravel, fairly
3.5 compact. Refusal.

T-5a

0.0
0.25 Black topsoil, sandy.
Brown, medium to fine
2.25 sand.
Ashes encountered.
2.3 Refusal.

T-6a

0.0
1.0 Black topsoil, sandy.
Brown fine sand becoming light gray with brown streaks at 8.0'.
10.0
Thin strata of very clean, uniform, coarse brown
10.17 sand and fine gravel.
Uniform, fine, brown sand stratified becoming wet at 16.0'.
17.0

T-7pa

0.0
Fill: Gravel and sand some ashes.
5.0

T-8a

0.0
Ash fill, very loose.
4.0

T-9a

0.0
1.0 Brown topsoil, sandy.
Compact ash fill to
2.83 thin loose ash
Fill - Refusal on decayed
2.83 auto cushion.

T-10a

0.0 Dark brown, fine to medium, organic,
1.0 sandy topsoil.
Light brown, clean,
2.5 fine sand.

T-11a

0.0 Dark brown, fine to medium, organic,
1.0 sandy topsoil.
Light brown, clean,
3.0 fine sand.

T-12a

0.0 Dark brown, fine to
1.0 medium, sandy topsoil.
Light brown, clean,
2.5 fine sand.

T-13a

0.0 Dark brown, fine to medium, organic,
1.0 sandy topsoil.
Light brown, clean,
2.5 fine sand.

T-14a

0.0 Dark brown, fine to medium, organic,
1.0 sandy topsoil.
Light brown, clean,
2.5 fine sand.

T-15a

0.0 Dark brown, organic,
0.2 sandy topsoil.
Ash, cinder and some
2.0 rubbish.

T-16a

0.0
Fill: Gravel with bricks, etc.
4.0

T-17a

0.0 Cinder fill
1.0 Refusal

T-18a

0.0
2.5 Topsoil
Dark brown, fine sand, uniform.
6.6
Light brown, fine sand.
11.0
12.5 Light gray, fine sand.

T-19a

0.0
3.0 Medium-coarse SAND
Ash and cinder FILL, occasional pieces of broken glass, etc.
8.0 Refusal.

T-20a

0.0 FILL (composed of mixture of clean and dirty sand with asphalt coated crushed stone.
5.0 Refusal.

T-21a

0.0
0.2 Black organic topsoil.
Cinder, ash and tin can FILL.
11.0 Refusal due to caving

T-22a

0.0
Cinder and rubbish FILL.
6.5
Black, dirty, medium coarse SAND.
14.0
Clean fine sand.
14.5 Refusal due to caving

T-23a

0.0
Brown, silty sand.
3.5
Light brown, clean, fine sand.
9.0
Fine sand and gravel.
11.0 Refusal.

T-24a

0.0
1.8 Black, silty, organic topsoil.
Light brown fine sand
9.0
Fine to medium sand and gravel.
19.5 Refusal.

T-25a

0.0
1.5 Black, sandy, organic topsoil.
3.0 Light brown silty sand.
Light and dark fine sand.
16.0

T-26a

0.0 Black, sandy, organic
0.2 topsoil.
Clean fine sand, slightly silty for first few inches, rusty near bottom.
16.0

T-27a

0.0 Black, sandy, organic
0.8 topsoil.
Clean fine sand.
16.0

T-28a

0.0 Black oil soaked sand mixed with rubbish.
3.0
Clean, fine sand.
13.0 Refusal.

T-29a

0.0 FILL (Rubbish, brick, concrete fragments, etc.)
6.0 Refusal

T-30a

0.0 FILL (Mostly fine to medium sand.)
4.0
Clean fine sand.
6.5
Dark brown, fine, silty sand
13.0 Refusal due to caving.

T-31a

0.0 1.0 Rubbish FILL.
Dark brown, silty fine sand.
4.0
Light brown, clean, fine sand.
11.0
Fine to medium sand and gravel.
12.0 Refusal.

T-32a

0.0 Dark brown, organic
1.0 topsoil.
Dark brown, silty,
3.5 fine sand.
Fine to medium sand with some gravel.
16.0

T-33a

0.0 Black, silty organic
2.2 topsoil.
Silty sand with a few
3.5 cobbles.
Clean, fine sand.
14.5 Refusal.

T-34a

0.0
3.0
Light brown clean sand.
13.0 Refusal.

T-35a

0.0 Black, sandy, slightly
1.5 organic topsoil.
Dark brown, slightly
4.0 silty fine sand.
Gray to dark brown fine to medium sand.
7.0 Refusal due to caving

T-36a

0.0 Dark brown, sandy,
2.5 organic topsoil.
Light brown, slightly silty, fine sand.
5.5
Fine sand and gravel.
6.0 Refusal due to caving

T-37a

0.0 Black, silty organic
1.0 topsoil.
Light brownish gray,
4.0 fine sand.
Gray fine sand with thin strata of peat.
5.0 Refusal due to caving

T-38a

0.0
2.0 Black, organic topsoil.
Fine to medium, clean sand.
10.0 Refusal due to caving

MERRIMACK VALLEY FLOOD CONTROL
SHAWSHEEN RIVER
LAWRENCE AND NORTH ANDOVER, MASS
RECORD OF EXPLORATION

IN SHEETS SHEET NO.

U. S. ENGINEER OFFICE, BOSTON, MASS.

APPROVAL RECOMMENDED: John L. Allen, Chief Engineering Division

SUBMITTED: Chief, Design Section, Engineering Division

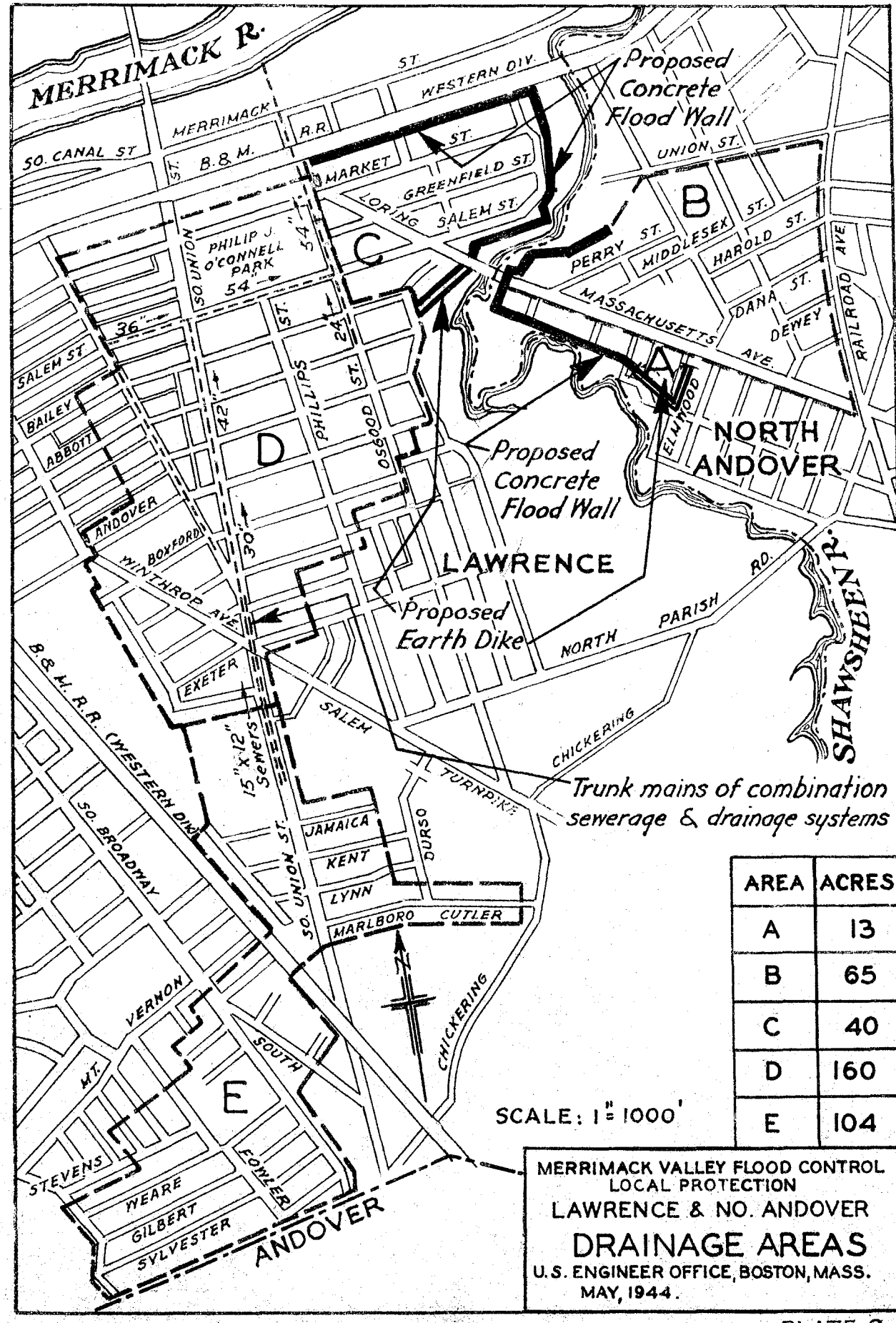
APPROVED: U.S. Colonel, Corps of Engineers, District Engineer

DES. BY W.L. Shannon, Engineer
DR. BY
TR. BY
CH. BY

FILE NO. M55-12/22

NO. | CHARACTER REVISIONS | DATE | BY

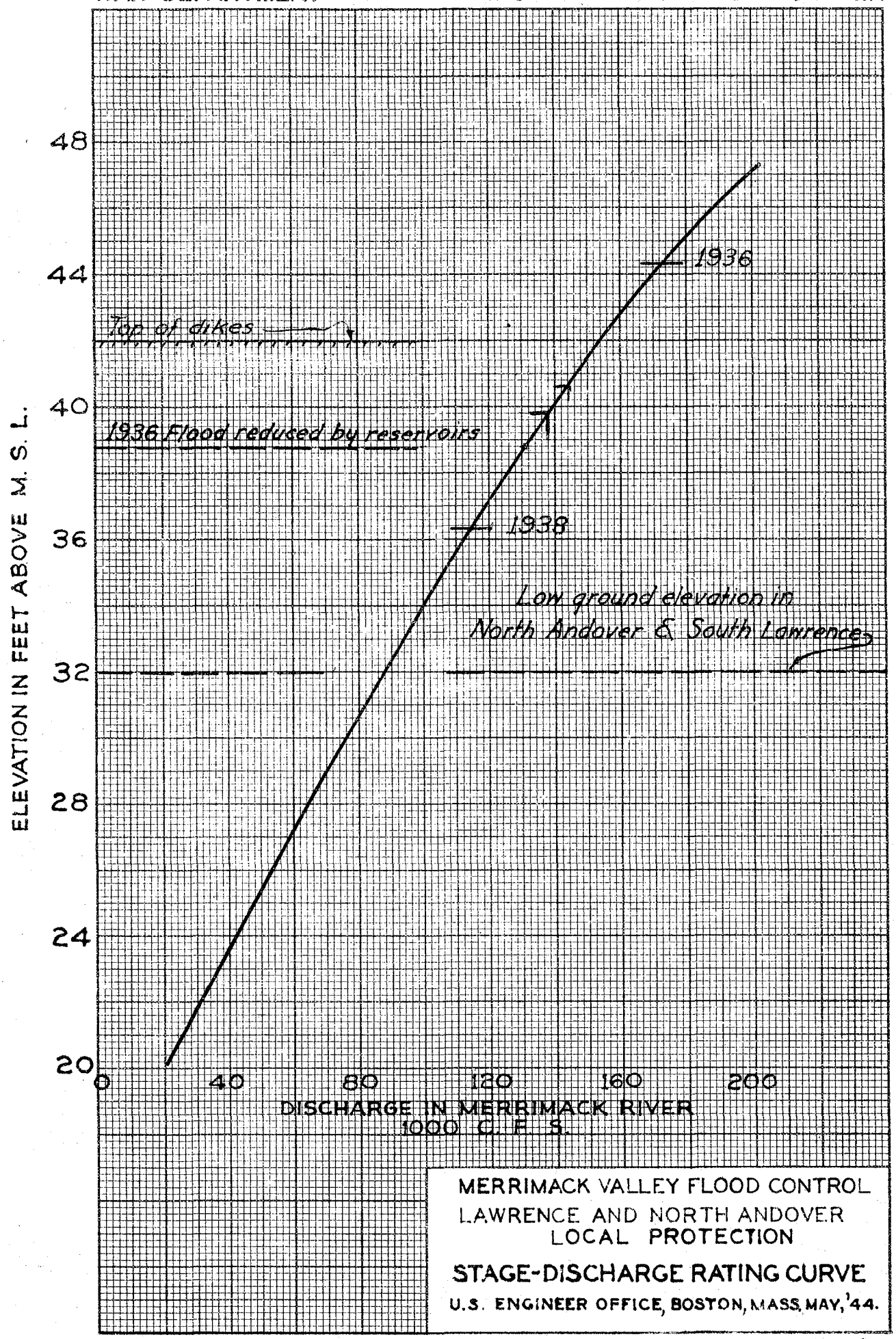
PLATE 7



AREA	ACRES
A	13
B	65
C	40
D	160
E	104

PLATE 8

WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY

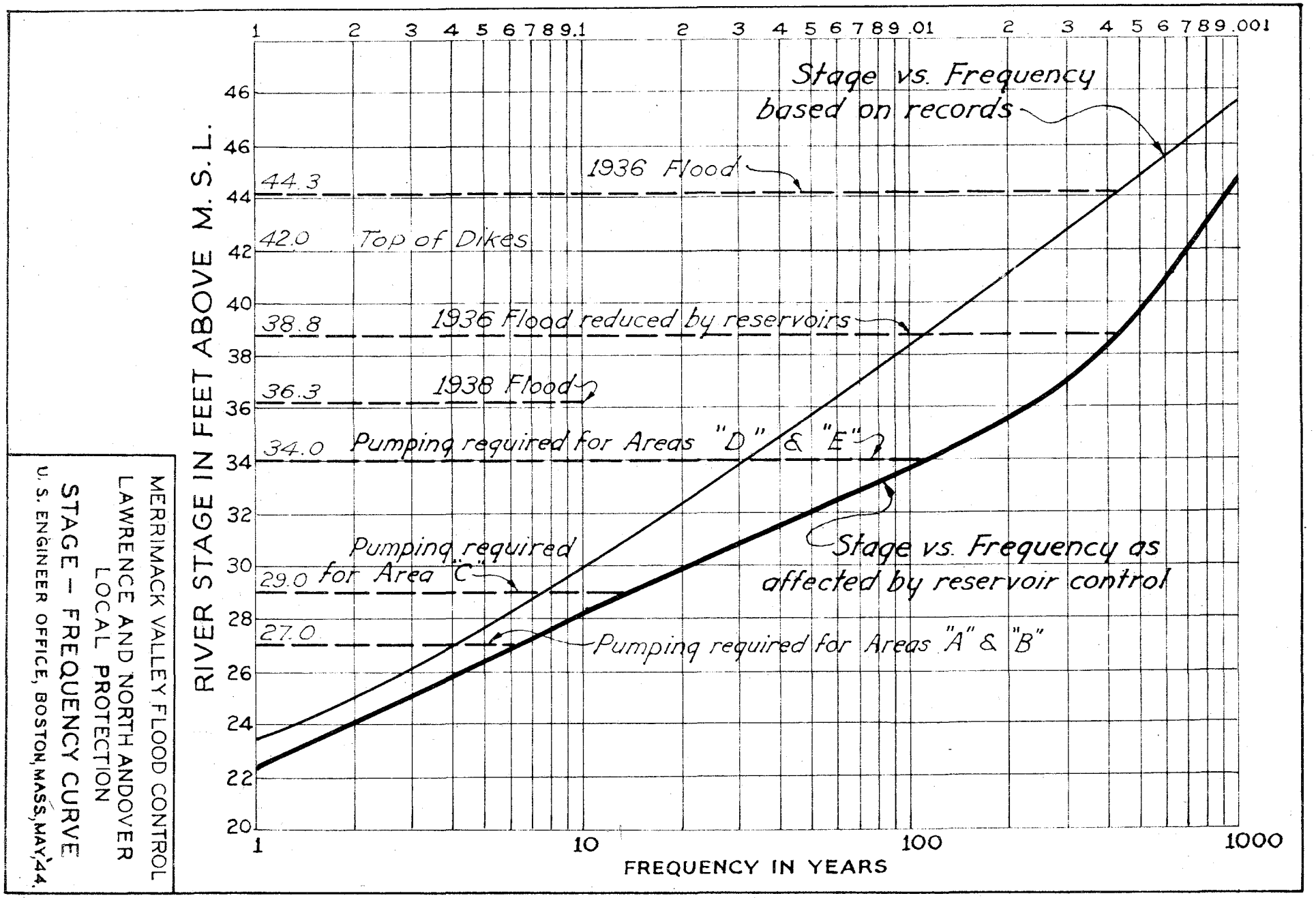


MERRIMACK VALLEY FLOOD CONTROL
LAWRENCE AND NORTH ANDOVER
LOCAL PROTECTION

STAGE-DISCHARGE RATING CURVE

U.S. ENGINEER OFFICE, BOSTON, MASS. MAY, '44.

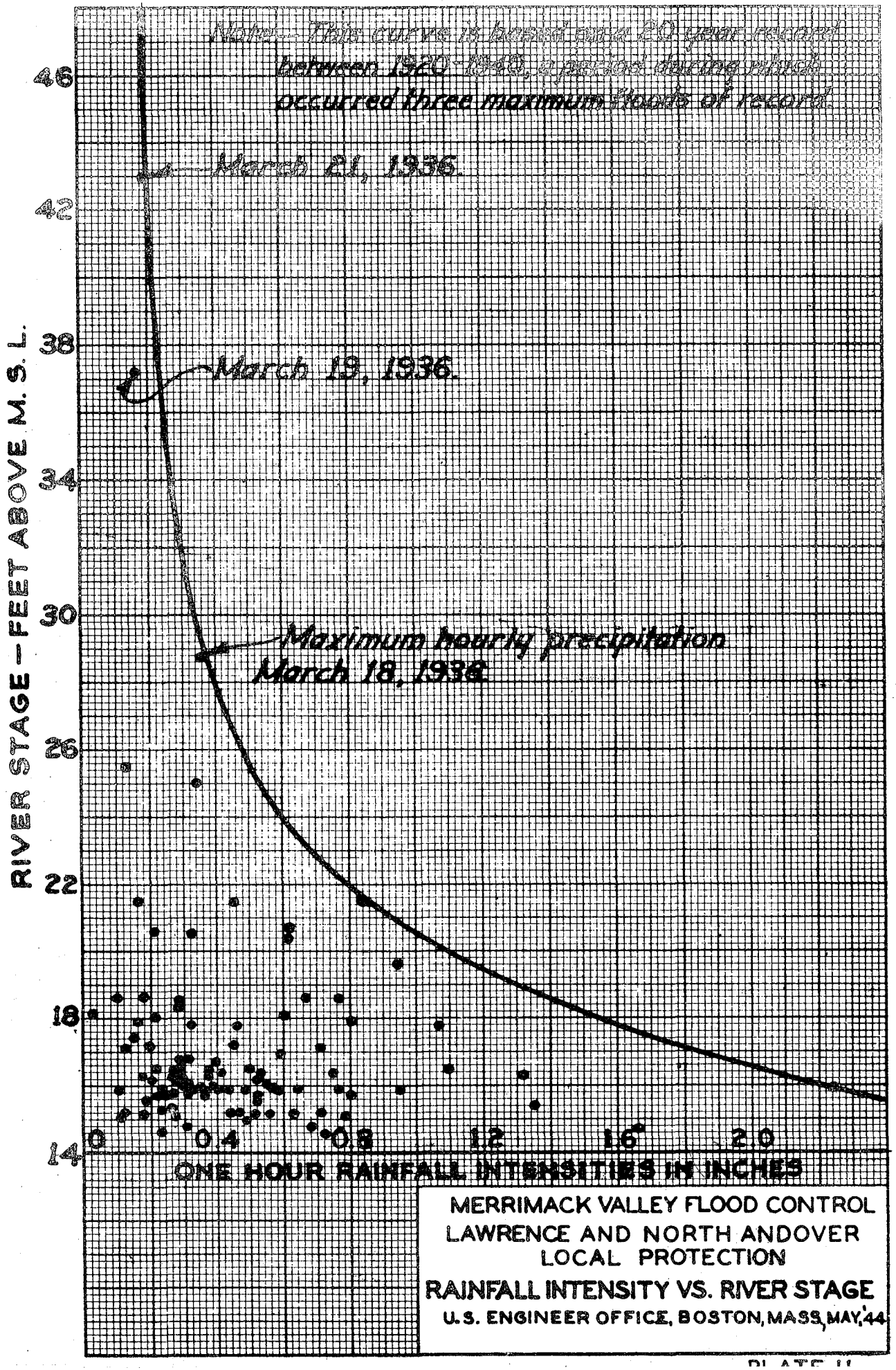
PLATE 2



MERRIMACK VALLEY FLOOD CONTROL
LAWRENCE AND NORTH ANDOVER
LOCAL PROTECTION
STAGE – FREQUENCY CURVE
U. S. ENGINEER OFFICE, BOSTON, MASS., MAY, '44.

PLATE 10

WAR DEPARTMENT CORPS OF ENGINEERS U.S. ARMY

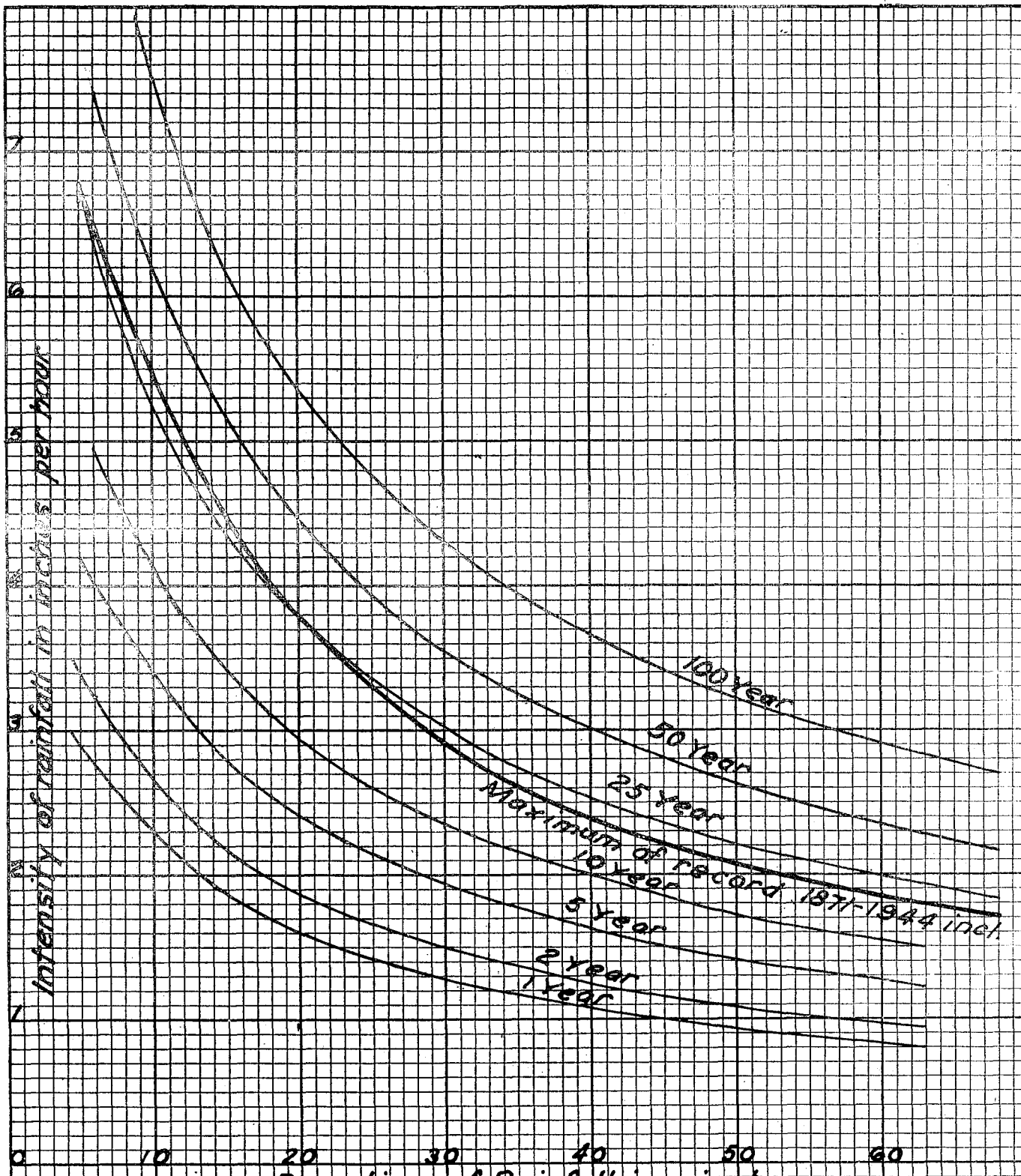


MERRIMACK VALLEY FLOOD CONTROL
LAWRENCE AND NORTH ANDOVER
LOCAL PROTECTION

RAINFALL INTENSITY VS. RIVER STAGE

U.S. ENGINEER OFFICE, BOSTON, MASS, MAY, '44

PLATE II



NOTE:

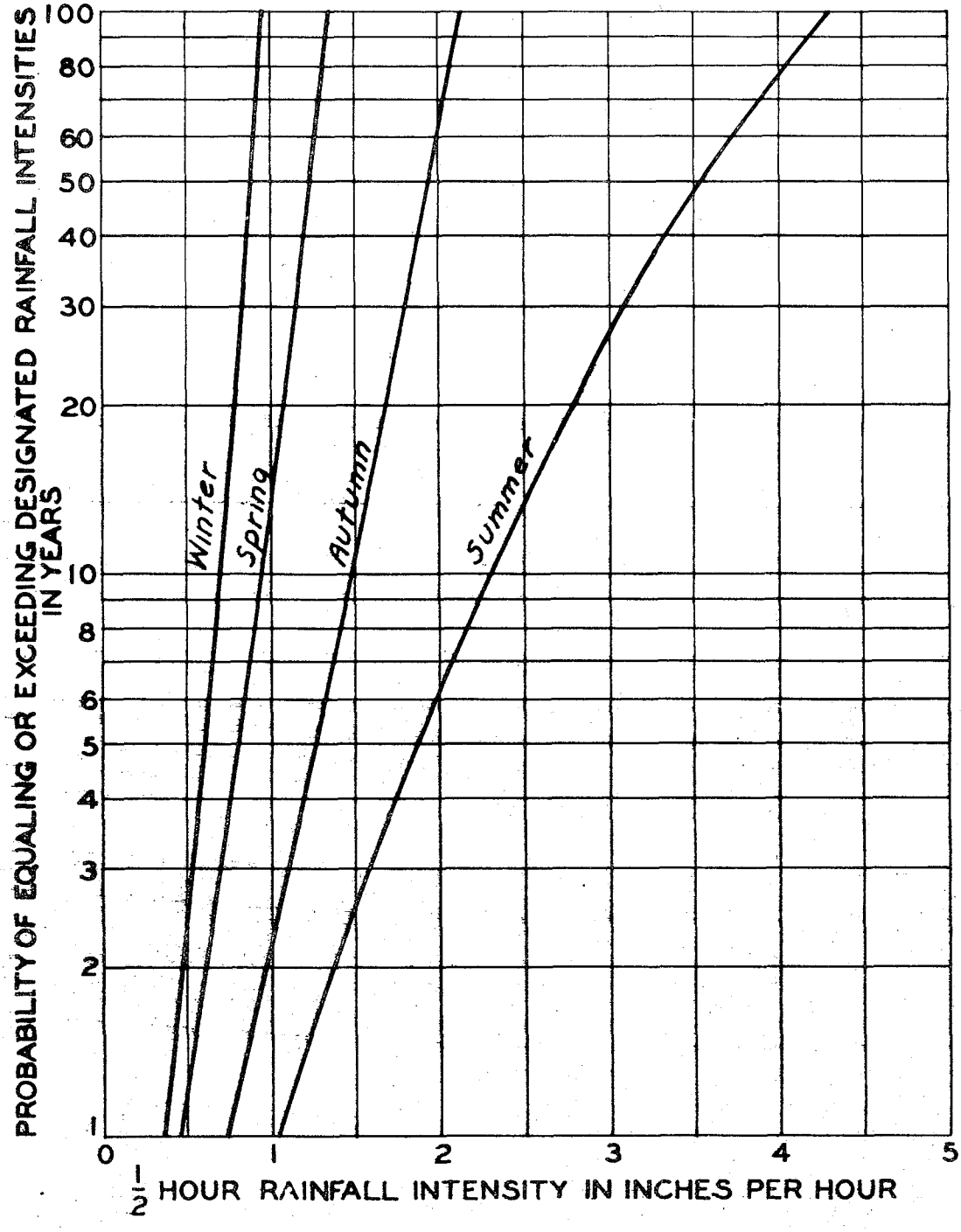
Curves by Charles W. Sherman, A.S.C.E.
Transactions 1931 - Pg. 951 Vol. 95
Records 1871-1930 incl. 1&2 yr. frequencies revised to include records thru 1944.

MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION
LAWRENCE & NO. ANDOVER

INTENSITY OF PRECIPITATION AT BOSTON, MASS.

U.S. ENGINEER OFFICE. BOSTON, MASS.

PLATE IIA



Note:

Rainfall intensities are based on records at Boston, Mass. for the period 1905-1940 incl.

Frequency of occurrence is based on the formula, $Y = \frac{M}{N-0.5}$ *in which*

Y = Frequency of occurrence in years.

M = Period of record in years.

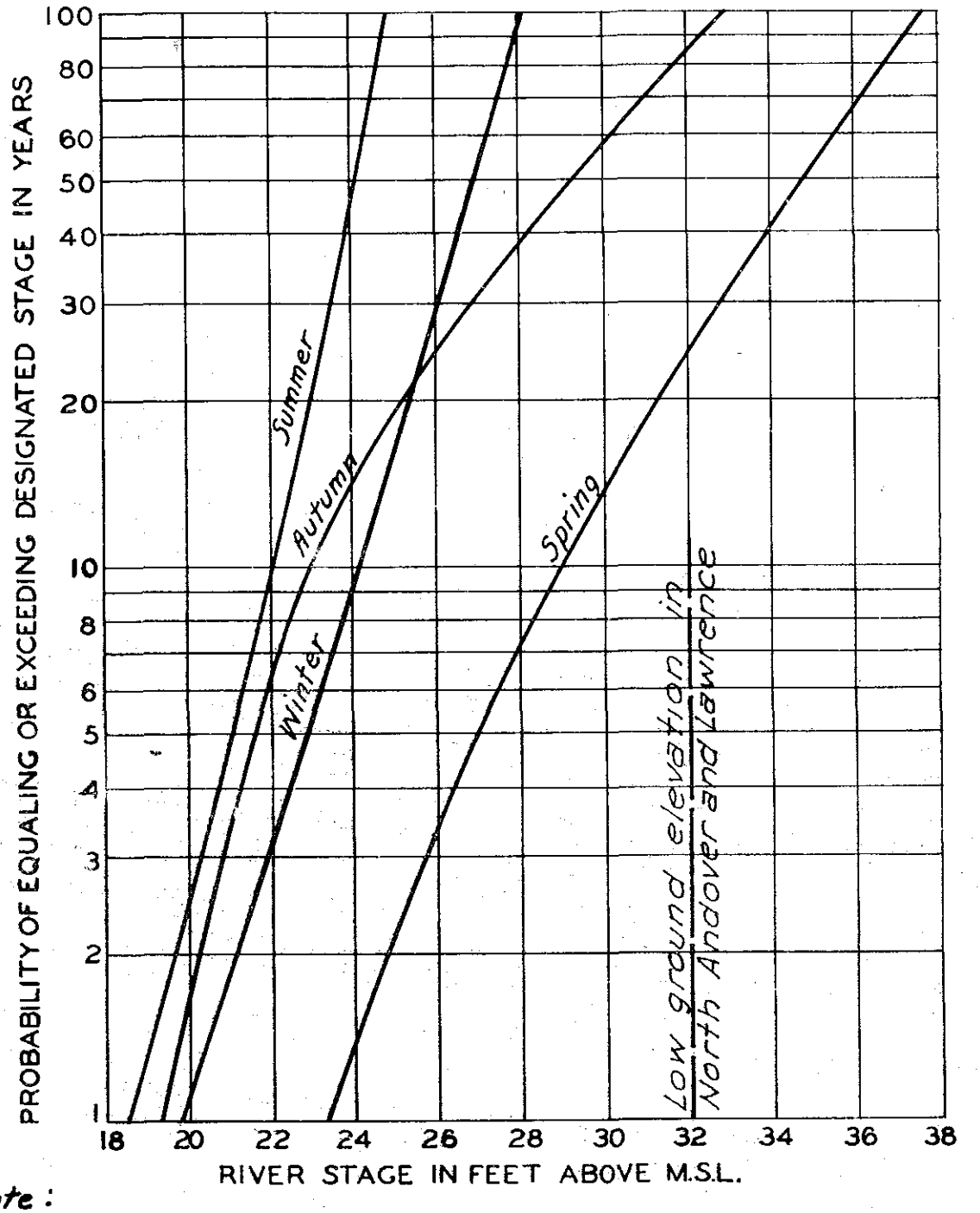
N = Order of magnitude of rainfall intensity.

MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION
LAWRENCE & NO. ANDOVER, MASS.

SEASONAL FREQUENCY CURVES
½ HOUR RAINFALL INTENSITY

U. S. ENGINEER OFFICE BOSTON, MASS.

PLATE 11B



Note :

River stages are based on the records at Lawrence, Mass. for the period 1849-1938 incl. and are not adjusted for the effect of existing or proposed flood control reservoirs.

Frequency occurrence is based on the Formula $Y = \frac{M}{N-0.5}$ *in which*

Y = Frequency of occurrence in years.

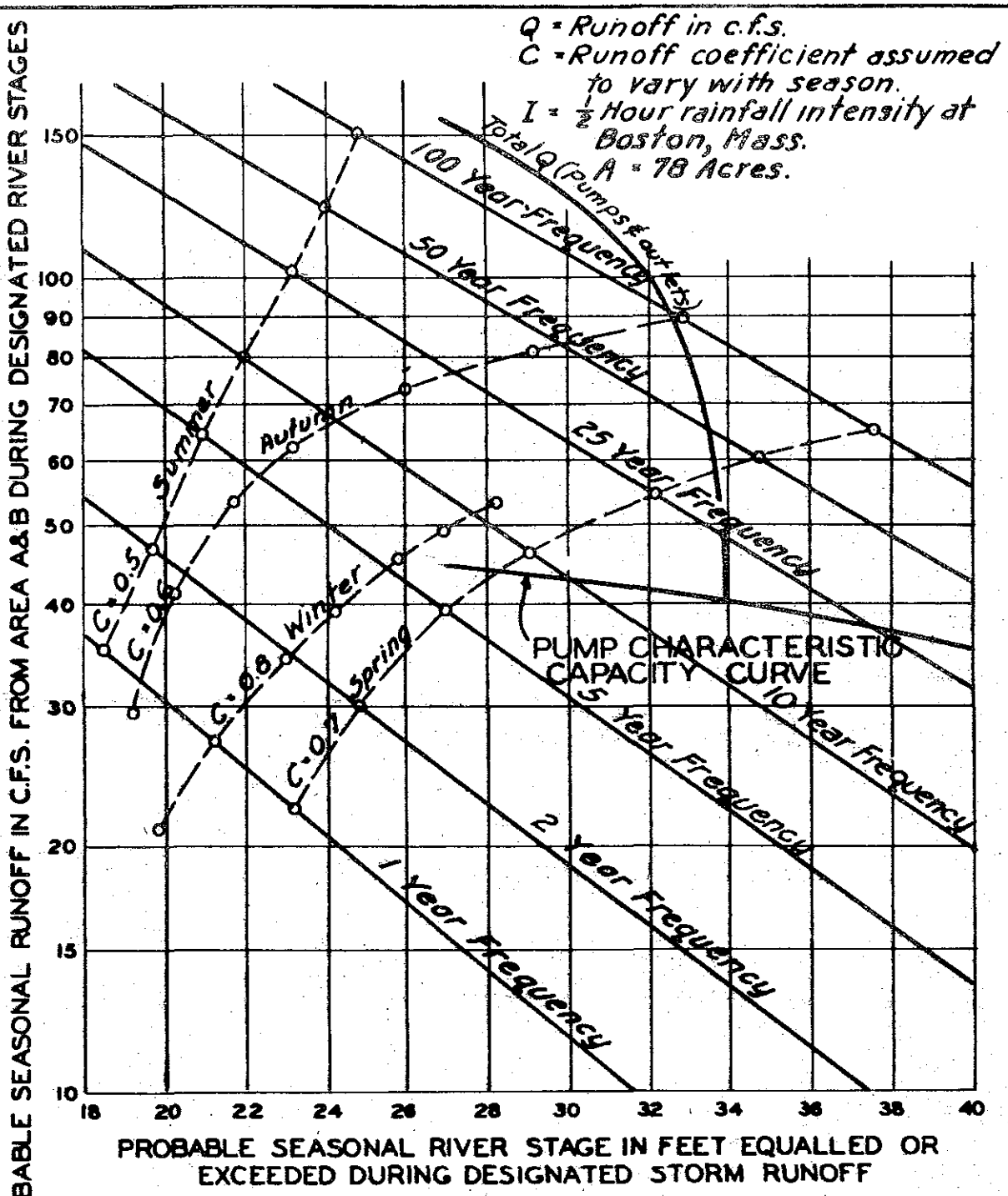
M = Period of record in years.

N = Order of magnitude of river stage.

MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION
LAWRENCE & NO. ANDOVER, MASS.
SEASONAL FREQUENCY CURVES
MERRIMACK RIVER STAGES

U. S. ENGINEER OFFICE BOSTON, MASS.

PLATE IIC



NOTES:

Runoff is based on the Rational Formula Q=CIA. (see Appendix A-1-d) River Stage is based on records by The Essex Co. at Lawrence, Mass. The effect of existing and proposed flood control reservoir were not considered in this study.

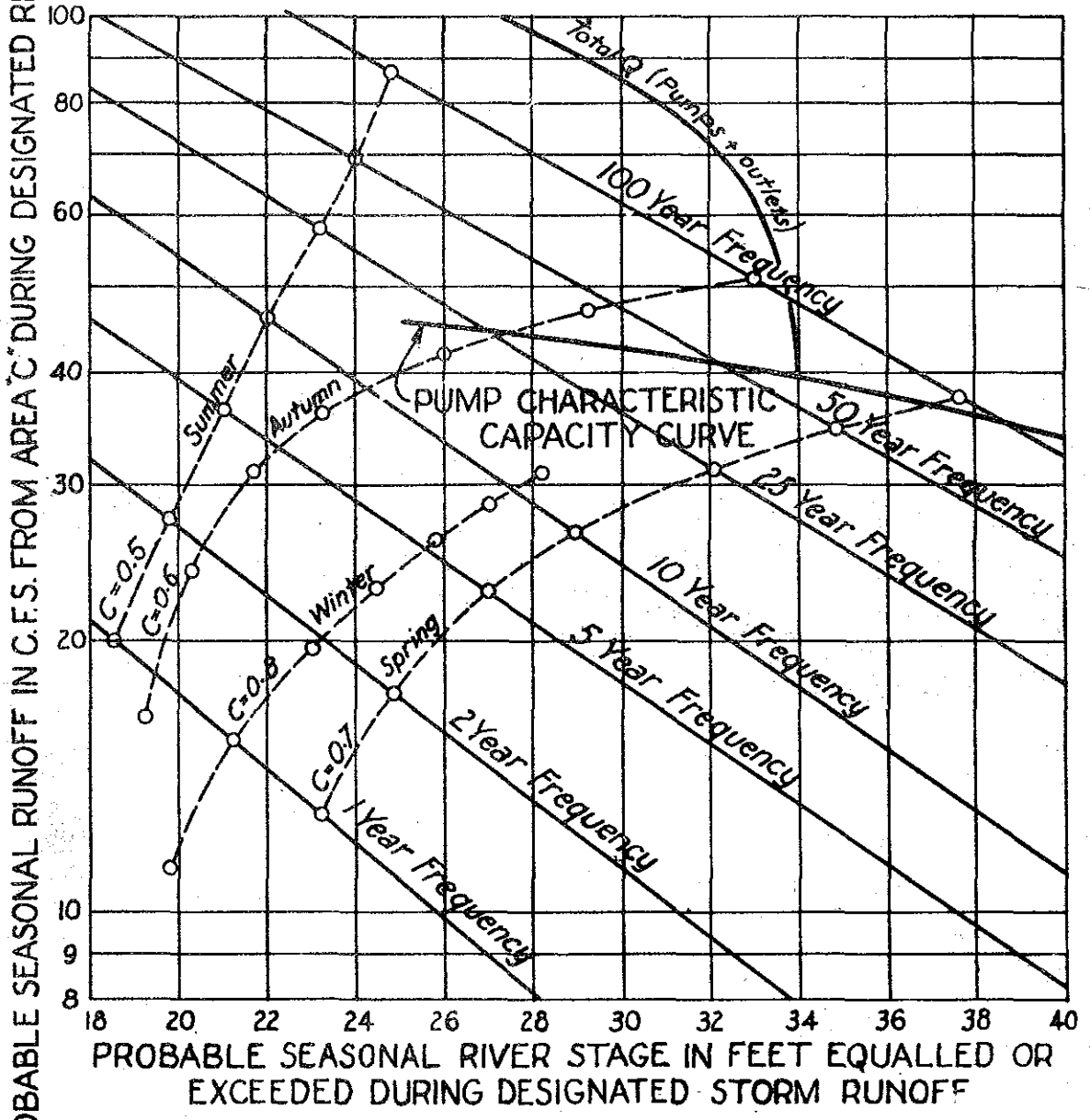
MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION
NORTH ANDOVER, MASS.

SEASONAL FREQUENCY CURVES
RUNOFF VS RIVER STAGES

U. S. ENGINEER OFFICE BOSTON, MASS.

PLATE IID

Q = Runoff in c.f.s.
C = Runoff coefficient assumed to vary with season.
I = ½ Hour rainfall intensity at Boston, Mass.
A = 40 Acres.



NOTES:

Runoff is based on the Rational Formula $Q = CIA$. (see Appendix A-1-d.) River Stage is based on records by The Essex Co at Lawrence, Mass. The effect of existing and proposed flood control reservoir were not considered in this study.

MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION
SOUTH LAWRENCE, MASS.

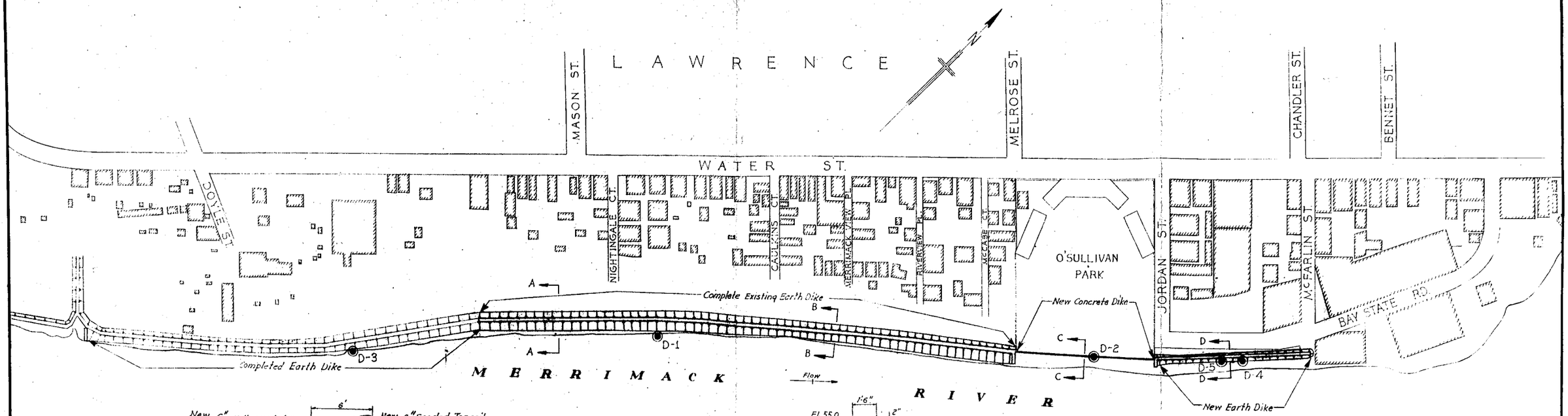
SEASONAL FREQUENCY CURVES
RUNOFF VS RIVER STAGES

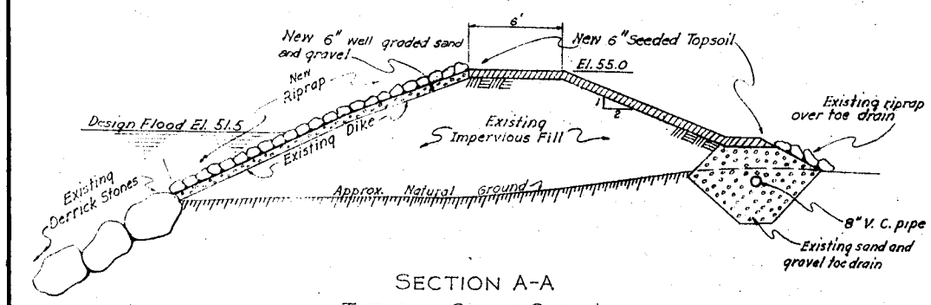
U. S. ENGINEER OFFICE BOSTON, MASS.

PLATE IIE

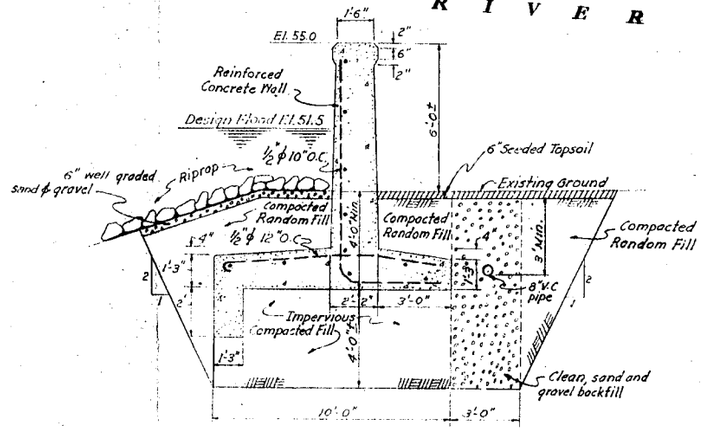
WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

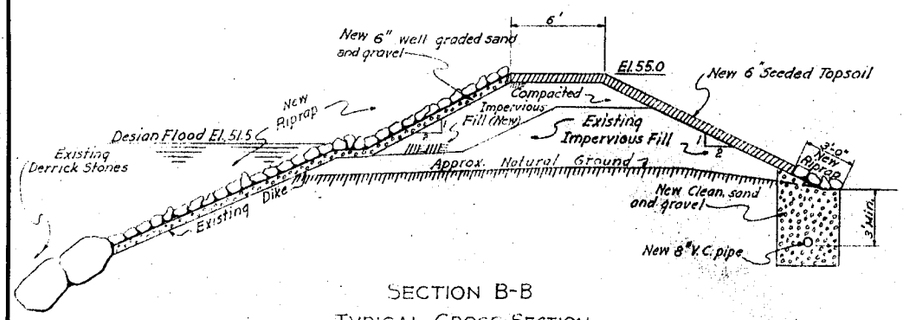




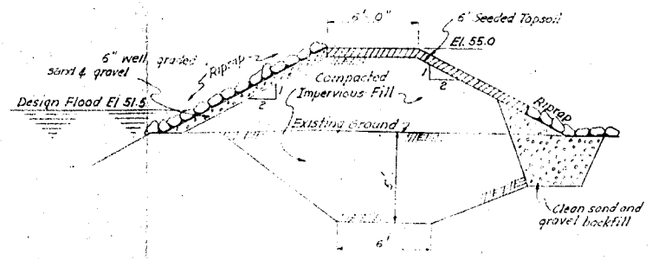
SECTION A-A
TYPICAL CROSS SECTION
PARTIALLY COMPLETED DIKE
0 1 5 10
SCALE OF FEET



SECTION C-C
TYPICAL CROSS SECTION
NEW CONCRETE DIKE
0 1 5 10
SCALE OF FEET



SECTION B-B
TYPICAL CROSS SECTION
PARTIALLY COMPLETED DIKE
0 1 5 10
SCALE OF FEET



SECTION D-D
TYPICAL CROSS SECTION
NEW EARTH DIKE
0 1 5 10
SCALE OF FEET

Note:— Indicated stage of the design flood is based on the March, 1936 flood, reduced by the comprehensive reservoir system. Earth dikes and concrete walls are structurally designed with flood stage at top elevation.

RECORD OF EXPLORATION

D-1

0.0	Loose, fairly uniform fine to medium SAND, with occasional small pebble. (Probable fill material.)
3.5	
8.0	Fairly soft, silty, fine SAND.
	Loose, slightly silty, fine to medium SAND.
26.5	Fairly compact, slightly silty, fine SAND with cobbles and numerous pebbles.
32.0	Compact, slightly silty, gravelly, fine to coarse SAND.
	Refusal - bottom of hole

D-2

0.0	FILL - slightly compact, fine SAND with pieces of ash and brick.
5.0	Fairly soft, fine SAND and coarse SILT.
18.5	Loose, variably gravelly, fine to coarse SAND with cobbles and boulders.
23.0	Fairly soft, slightly clayey, fine to coarse SILT.
24.0	Slightly compact gravelly medium to coarse SAND.
25.0	Fairly compact, silty fine SAND with occasional small seam of fine gravel.
41.0	Loose, slightly silty, fine to coarse SAND.
41.8	Soft, slightly sandy SILT.
42.0	Bottom of hole

D-3

0.0	Loose, fine to medium SAND. (Probable fill material.)
3.5	Slightly compact to soft variably slightly silty fine to medium SAND.
	Loose, slightly silty, gravelly, fine to coarse SAND with cobbles and boulders.
32.0	Fairly compact, slightly silty, fine SAND.
35.0	
40.5	Bottom of hole

D-4

0.0	Topsoil and fill
13.0	Loose, gray, fine, silty SAND.
35.0	Compact, gray, gravelly, fine to medium SAND.
39.5	Bottom of hole

D-5

0.0	Topsoil and fill (brick fragments, ashes, sand)
15.0	Loose, gray, slightly gravelly, silty, fine SAND. Decomposed wood
20.0	Bottom of hole

MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION MERRIMACK RIVER
LAWRENCE, MASS.
PRELIMINARY LAYOUT
SCALE OF FEET
0 100 500

U. S. ENGINEER OFFICE, BOSTON, MASS. APRIL, 1944

APPROVAL RECOMMENDED
Chief Engineering Division

APPROVED:
Colonel, Corps of Engineers
District Engineer

SUBMITTED:
Chief Design Section, Eng. Division

Asst. Engineer

NO.	CHARACTER	DATE	BY
1	Clean gravel changed to well graded sand & gravel	10-14-44	EJL

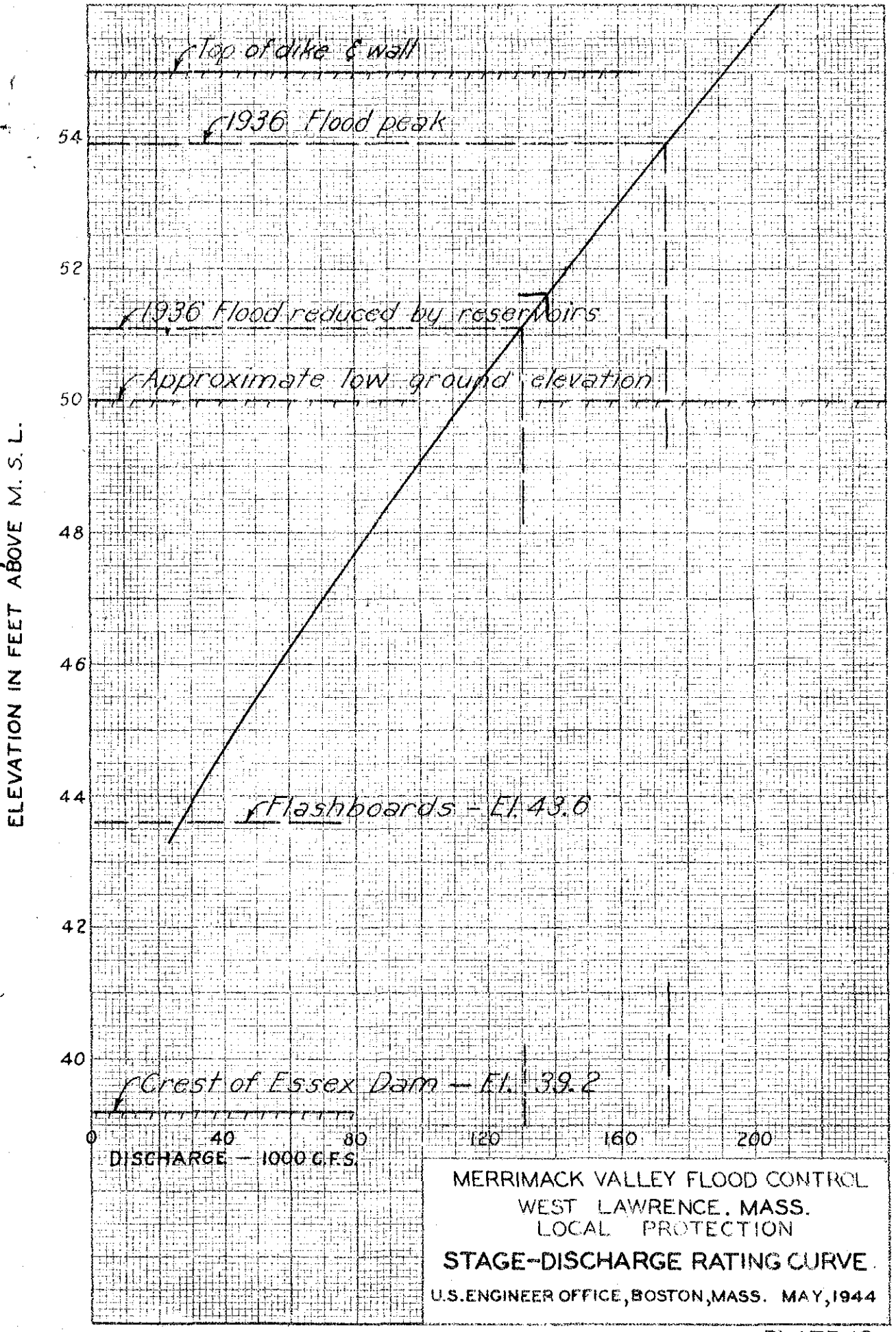
REVISIONS

DES. BY
DR. BY
TR. BY
CH. BY

FILE NO. NVS-12/23

PLATE 12

WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY



MERRIMACK VALLEY FLOOD CONTROL
WEST LAWRENCE, MASS.
LOCAL PROTECTION

STAGE-DISCHARGE RATING CURVE

U.S. ENGINEER OFFICE, BOSTON, MASS. MAY, 1944

PLATE 13

Original Copy

Superseded

Merrimack River Basin Flood Control

Definite Project Report

for

NORTH ANDOVER AND LAWRENCE, MASSACHUSETTS, LOCAL PROTECTION

Corps of Engineers, U. S. Army

U.S. Engineer Office — Boston, Mass.

DEFINITE PROJECT REPORT

NORTH ANDOVER AND LAWRENCE, MASSACHUSETTS, LOCAL PROTECTION

MERRIMACK RIVER BASIN FLOOD CONTROL

Table of Contents

TEXT OF REPORT

APPENDICES

Corps of Engineers, U. S. Army

U. S. Engineer Office Boston, Mass.

May 1944

WAR DEPARTMENT
United States Engineer Office
3d Floor, Park Square Bldg.
31 St. James Avenue
Boston 16, Mass.

22 May 1944.

Subject: Definite Project Report for North Andover and Lawrence, Massachusetts, Local Protection.

To: The Chief of Engineers, U. S. Army, Washington, D. C., Through the Division Engineer, New England Division, Boston, Massachusetts.

1. Project Authority.- The projects for local protection at North Andover and Lawrence, Massachusetts, as described herein, are proposed as part of the comprehensive plan for flood control reservoirs and related flood control works for the Merrimack River Basin authorized by the Flood Control Acts approved 22 June 1936 and 28 June 1938. The Flood Control Act of 1938 provides that, "The project for flood control in the Merrimack River Basin as authorized by the Flood Control Act approved 22 June 1936, is modified to provide, in addition to the construction of the system of flood control reservoirs, related flood control works which may be found justified by the Chief of Engineers".

2. Previous Investigations.- The investigations and studies upon which the authorized project for the Merrimack River Basin is based are described in House Document No. 689, 75th Congress, 3rd Session.

a. The local protection work for North Andover was contemplated in 1940 and reference is made to the "Survey Report for Navigation, Flood Control, and Water Power, Merrimack River, Massachusetts and New Hampshire", dated 1 April 1940 and submitted to the Chief of Engineers. This report notes that the local flood protection works were under consideration and presents an approximate estimate of the proposed construction costs. Reference is made also to the "Subproject for North Andover, Merrimack River Basin, Mass.", dated 22 August 1940, submitted to the Chief of Engineers.

b. The City of Lawrence planned to sponsor two projects under the Works Progress Administration in 1939:

(1) The construction of dikes and walls in south Lawrence along the west bank of the Shawsheen River, similar to the layout shown on Plate 5, and

(2) The construction of dikes and walls in west Lawrence along the north bank of the Merrimack River, similar to the plan shown on Plate 12.

Work on the south Lawrence project was not started due to the uncertain conditions of that period. The west Lawrence project was initiated but only partially completed before labor curtailments forced the construction to terminate. The completed portion of the dike affords no protection in its present condition.

3. Location and Description of Areas Affected.-

a. General.- (1) North Andover and south Lawrence.- The proposed projects are located on the banks of the Shawsheen River which forms the boundary between the City of Lawrence and the Town of North Andover. The sites of the dikes and walls are about one-half mile upstream from the confluence of the Shawsheen and Merrimack Rivers which is 27.3 miles above the mouth of the Merrimack River and approximately 24 miles north of Boston, Massachusetts. The area requiring protection in North Andover is a residential development of approximately 25 acres, with an estimated population of about 500. The total valuation is estimated to be $500,000. The area requiring protection in south Lawrence is a residential development of approximately 40 acres with a population of approximately 1000. The total assessed valuation of the south Lawrence area in 1939 was $770,000.

(2) West Lawrence.- The incomplete W.P.A. project in west Lawrence is located on the north bank of the Merrimack River just upstream from the Essex Dam, or approximately 29.2 miles above the mouth of the Merrimack River. The area subject to damage during past floods consists of approximately 125 acres and is thickly developed with commercial establishments and residences. The total assessed valuation in 1939 was $1,760,000.

b. Flood Conditions.- (1) North Andover and south Lawrence.- All floods that cause damage at these two sites are the result of backwater from flood stages on the Merrimack River and are not produced by flood flows on the Shawsheen River. The Shawsheen River has a total drainage area of 73 square miles above its mouth and is a sluggish stream, its flow controlled by numerous ponds and swamps. The greatest flood of record occurred in March 1936 when the areas in North Andover and south Lawrence were flooded to a maximum depth of about 15 feet. The estimated direct losses totalled $280,000 in North Andover and approximately the same in the south Lawrence area to be protected by this proposed project. Parts of both areas were

flooded again during the hurricane flood in September 1938.

(2) West Lawrence. Flood damage in this area is the result of the backwater head produced by the Essex Dam. In the 1936 flood the area was inundated to a maximum depth of about 4 to 5 feet and in 1938 a smaller area was affected with maximum depths ranging from 1 to 2 feet. The 1936 flood caused estimated direct losses of $165,000.

c. Current Flood Control Projects.- The comprehensive plan for flood control in the Merrimack River Basin and its present status is illustrated on Plate 1. Two reservoirs have been completed: (1) Franklin Falls controlling the Pemigewasset River, the principal tributary of the Merrimack River, and (2) Blackwater Reservoir on the Blackwater River, a tributary of the Contoocook River. Other proposed reservoirs for controlling the Contoocook River are Mountain Brook, West Peterboro, Bennington and Beards Brook. The effects of these reservoirs, both existing and proposed, have been considered in determining grades for the walls and dikes to protect the inundated areas. Local protection projects also have been completed at Fitchburg, Lowell and Haverhill, Massachusetts. There are no other flood control works, existing or contemplated, which affect the projects described herein.

4. Definite Project Plan.-

a. Work Proposed.- The areas to be protected and the proposed plans for preventing damage from future floods are shown on Plates 3, 4, 5 and 12. All the areas are low and subject to inundation during the flood stages on the Merrimack River. It is proposed to provide protection similarly in all areas by constructing earth dikes and concrete walls. In North Andover and south Lawrence, it is necessary to provide pumping facilities for the sewage and drainage at each site. More detailed description of the proposed construction in the different areas are as follows:

(1) North Andover.- Approximately 2300 feet of concrete wall and 750 feet of earth dike are proposed in this section. New drains will be installed adjacent to the wall and dike to intercept surface run-off, and a pumping station will be constructed to pump both sewage and storm drainage from the local drainage area during flood periods.

(2) South Lawrence.- Approximately 3250 feet of concrete wall and 570 feet of earth dike are proposed in this section. New drains, interceptors, and diversion pipe will be installed to prevent flooding by backwater from the combined sewage and drainage system. A pumping station will be constructed to discharge the sewage and drainage during flood periods.

(3) West Lawrence.- The work in the west Lawrence area consists of completing the W.P.A. earth dike. The original W.P.A. plans specified an earth dike approximately 2300 feet long on the north bank. About 1000 feet of this dike was completed in the 1940 construction period, but the remainder of the dike requires riprapping, gravel toe drain, topsoil and seeding. The low area east of this earth dike will be protected by a concrete wall extending 350 feet along the river side of the O'Sullivan Park and a low earth dike approximately 375 feet in length. No piping or pumping station is required for this section.

b. Design Flood.- The greatest flood of record on the Merrimack River occurred in March 1936, when the discharge at Lawrence was 174,000 c.f.s. This flood exceeded all previous floods on the Merrimack River in an official record of 90 years and an unofficial record of nearly 200 years by 66,000 c.f.s. and 9 feet in stage. The flood in September 1938 had a peak discharge of about 120,000 c.f.s. and is the second highest flood of record. All flood flows on the Merrimack River are produced by either, or a combination of, heavy rainfall and melting snow in the headwaters of the Merrimack River Basin. The drainage areas of all tributary rivers in the south half of the basin are of such characteristics and size that, although they contribute to and augment the Merrimack River flow, they are not conducive to producing damaging floods on the Merrimack River. Consequently, as the maximum discharges are produced by the headwater tributaries in the mountainous portion of the drainage area, the maximum crest of the flood at Lawrence occurs about 55 hours, or more than two days, following the flood producing rainfall. This characteristic of the river hydraulics is noted because of its importance in the selection of pumping station capacities. It was concluded that the maximum flood of record, reduced by the comprehensive reservoir program, would provide a safe design criteria for local protection works in this area. This is the same design criteria for the completed local protection projects on the Merrimack River. The maximum river stages of record, both uncontrolled and as reduced by the proposed comprehensive reservoir system are shown on Plates 9 and 13 with the selected grades for the concrete walls and earth dikes. At North Andover and south Lawrence projects, a freeboard of 3 feet above the maximum stages of the 1936 floods as reduced by the reservoirs is provided for the sections of earth dike and a freeboard of 2 feet is provided for the concrete walls. The grades established are elevation 42.0 for the earth dikes and elevation 41.0 for the concrete walls. At the west Lawrence project the top of dike and concrete wall will be maintained at elevation 55.0, which corresponds to the grade previously established for the W.P.A. project. This grade provides 4 feet of freeboard over the peak stage of the 1936 flood reduced by reservoirs. This additional freeboard is considered essential for this location because the dike borders the main river channel and comparatively high velocities of flow are anticipated. In North Andover and south Lawrence

serious wave action will not occur for there will be no velocity of flow since the high stages in these areas are the result of backwater from the Merrimack River and the fetch, causing waves by wind velocities, is comparatively small. It is believed that these freeboard provisions are adequate allowance for (1) wave action, (2) degree of accuracy in reach-routing from the headwater tributaries to Lawrence, and (3) effective protection against the design flood until such time as the comprehensive reservoir system is completed.

5. Hydrology.- (See Appendix "A" for complete hydrological analysis.)

a. North Andover.- A 24-inch drain discharging into the Shawsheen River and a 24-inch trunk sewer that extends to the Merrimack River are located in the area affected by the proposed flood wall. (See Plate 5). The 24-inch drain with a discharge capacity of 20 c.f.s. serves a drainage area of 65 acres. An area of 13 acres between Massachusetts Avenue and the flood wall on the west will be provided with new drains and catchbasins. The 24-inch sanitary trunk sewer, with a capacity of 6 c.f.s. serves a comparatively small area at present, but was installed for the purpose of serving 1.5 square miles of North Andover, if the development of the town warranted its extension. It is proposed to construct a pumping station east of Massachusetts Avenue as an integral part of the concrete flood wall to pump the sanitary sewage and storm run-off into the Shawsheen River during high river stages. Based on a storm of a ten year frequency, and including sewage flow and seepage, the computed maximum discharge from the entire drainage area of 78 acres is 51 c.f.s.. To satisfy existing and proposed drainage and sewerage systems which have a total capacity of 36 c.f.s. it is proposed to provide pumps with a total capacity of 50 c.f.s. The maximum operating head will be about 25 feet against the river stage produced by the design flood.

b. South Lawrence.- A 54-inch diameter brick sewer flowing north in Osgood Street serves as a trunk outlet for both sanitary sewage and storm drainage in Areas "C" and "D" and for sanitary sewage from Area "E" (Plate 8). The drainage areas are as follows:

Area "C"	-	40 acres
Area "D"	-	160 acres
Area "E"	-	104 acres
Total Area	-	304 acres

Area "C" is the low area inclosed by the proposed dike and is the approximate limit of the section inundated during the past floods. Areas "D" and "E" are at higher elevations and are not subject to flooding except during extreme high floods. The discharge capacities

of the sewers in Area "C" are approximately 14.2 c.f.s. compared with a computed discharge of 31 c.f.s. based on a 30 minute rainfall for a ten year frequency. The capacity of the 54-inch trunk sewer is approximately 100 c.f.s. compared with a maximum computed run-off and sewage flow of approximately 110 c.f.s., On the basis of frequency studies of Merrimack River flows, (Plate 10), it is expected that pumping will be required once every 12 years in Area "C", but in Areas "D" and "E", it is anticipated that pumping will be required only once in 110 years. New pipes and control facilities will be provided to take advantage of the difference in ground elevation and the probability of pumping requirements by separating the flow in Area "C" from "D" and "E". It is proposed to provide pumps similar to the Andover project with a total capacity of 50 c.f.s.. The maximum design pumping head will be approximately 25 feet.

c. West Lawrence.- A hydrological analysis is not required for west Lawrence. The area is served by a combination sewerage and drainage system that discharges into the Merrimack River downstream from the Essex Dam, and consequently, is unaffected by backwater. During the initiation of the project, the City of Lawrence installed new drains in the area inside and adjacent to the dike to prevent local ponding. The construction of the dike and wall does not effect the present sewerage and drainage systems, and consequently, interceptors and a pumping station are not necessary. A toe drain will be included to intercept subsurface seepage during flood stages, with the flow discharging into the combination sewer system.

6. Engineering Design.-

a. Surveys.- A topographic survey was made of the North Andover flood area which included the immediate northeast bank of the Shawsheen River, its primary flood plain or "bottom land" and the northeast valley wall, together with land immediately adjacent to the top of the bank. Topographic data for the south and west Lawrence flood area sites were obtained from plans based on surveys made by the City of Lawrence in connection with the proposed W.P.A. projects for flood protection for these areas which were prepared several years ago. Additional information concerning the west Lawrence dike was obtained by a topographical survey to determine the scope of the work required to extend and complete the project initiated by the W.P.A. in 1940. These data are considered satisfactory for preliminary planning of the structures now proposed. Considerable use was made also of aerial photographs for location of topographical features and buildings in all areas.

b. Foundation Exploration.-

(1) North Andover and South Lawrence.- The foundation areas for the proposed dikes and retaining walls were

explored by three (3) drill holes and thirty-eight (38) auger borings. Overburden exposures were inspected and local residents questioned regarding soil conditions. The location of the explorations are shown on Plate 5 and the logs of borings taken at North Andover and south Lawrence are included on Plate 7. In general both projects are located on the top of the banks forming the valley walls of the Shawsheen River. The banks are covered over large areas by a surficial deposit of rubbish from one foot to several feet in depth. There are three large rubbish and ash dumps and the extent of these is shown on Plate 5. In general, these dumps were constructed by end dumping from the original valley walls, thus they extend in depth to the elevation of the present river flood plain. Beneath the rubbish fills, stratified sands and silts with some gravel phases extend to depths of 30 to 50 feet below the river flood plain. Bedrock, a granite gneiss, was encountered at drill hole D1 at a depth of 27.5 feet below the river flood plain. The rubbish and ash fill is exceedingly non-uniform but very pervious, while the underlying natural deposits have coefficients of permeability as determined from remoulded samples, as follows:

Soil	Range of Coef. Of Permeability cm. per sec.
Clean Sands and Gravelly Sands	10^{-3} to 10^{-4}
Silts	10^{-5} to 10^{-6}

(2) West Lawrence.- The foundation areas for the proposed dikes and retaining wall were explored by five (5) drill holes and six (6) test trenches. Overburden exposures were inspected. The dike construction carried out by the W.P.A. was observed and tested for compaction and permeability. The locations and logs of the drill hole explorations made are shown on Plate 12. In general, the existing portion of the dike constructed by the W.P.A. and the unconstructed portion of the dike to the southwest corner of O'Sullivan Park are and will be founded upon a deposit of loose sand with variable fine and coarse phases and occasional pockets of clayey silt. The fill encountered at the ground surface in drill holes D1 and 3 is the existing W.P.A. embankment. In the area of the concrete dike and the earth dike to the northeast, a deposit varying from 5' to 15' of fill was encountered in drill holes D2, D4, and D5. The fill is very variable and typical of unassorted rubbish, ash and earth fills. Beneath the fill, fine sand was encountered with variable fine and coarse phases. Bedrock at the site is believed to be at considerable depths but is outcropping on the south bank of the Merrimack River opposite the dike. The W.P.A. constructed the existing portion of the earth dike using predominately a fairly pervious silty sand and gravel obtained as stripping from a glacial gravel deposit. A small portion of the dike at the northeast end of the constructed portion was built of

an impervious, well graded clayey sand and gravel (glacial till). The toe trench was constructed partly of washed gravel screenings 2" to 3" in size and partly of clean sandy gravel. The coefficients of permeability of the various soils encountered in the drill holes and used for construction are as follows:

Material	Coefficient of Permeability cm. per sec.
Fine sand foundation of dike	8 to 24 x 10^{-4}
Silty sand and gravel embankment fill as placed by W.P.A.	6 to 40 x 10^{-4}
Well graded clayey sand and gravel fill as placed by W.P.A.	.6 x 10^{-4}

c. Description of Walls and Dikes.- (1) North Andover.- The protective structures for the North Andover area include an earth dike approximately 750' in length, and a reinforced concrete flood wall approximately 2300' in length. The earth dike will border the east line of Elmwood Street starting from existing ground elevation 42.0 and extending southwesterly approximately 350' to the end of the street where it will make a right angle and continue approximately 400' northwest to join the end of the proposed reinforced concrete flood wall. Space limitations prevent extension of the earth dike section beyond its juncture with the proposed flood wall. The flood wall, top elevation 41.0, will extend northwesterly along the top of the valley bank for approximately 1150' then angle northeast, crossing Massachusetts Avenue and extending along the top of the bank to meet the existing grade at elevation 41.0. The flood wall is designed as a modified T section which will be of constant cross section throughout its entire length. For basis of design it is assumed that the flood stage is at the top of the wall (Elevation 41.0). The earth work supplementing the wall is varied to meet three different types of ground conditions. These three general types of ground conditions encountered at different places under the wall site are (1) existing ash and rubbish fill up to 14' in depth, (2) a compacted earth section built across a low area, and (3) firm existing natural ground. Where conditions (1) and (2) are encountered, the wall is to be constructed on a core of compacted impervious material extending vertically down through the pervious materials and into firm existing natural soil to a depth of at least 3 feet. Throughout the entire length of the flood wall and of the earth dike, provision has been made for drain lines to receive and carry off surface drainage and seepage. A removable

bulkhead to close the Massachusetts Avenue opening in times of flood, and facilities for storing the bulkhead when not in use will be provided. Plans and typical sections of the reinforced concrete flood wall and of the earth dike are shown on Plates Nos. 5 and 6. Channel changes in the existing channel of the Shawsheen River are contemplated to eliminate the possibility of riparian erosion which might undermine the proposed flood wall. This work will involve the construction of approximately 800 lineal feet of new channel including the excavation of approximately 11,000 cubic yards of earth from this new channel and placing it to fill the abandoned channel.

(2) South Lawrence.- The proposed system of protective structures in the south Lawrence area includes an earth dike approximately 570 feet in length and a reinforced concrete flood wall of varying cross section, approximately 3250 feet in length. The earth dike will extend from existing ground elevation 42.0 near Shawsheen Road northeasterly along the top of the valley bank to Loring Street where a concrete retaining wall parallel to the street line will retain the earth section. A removable bulkhead to close the Loring Street opening during periods of flood, and facilities for storing the bulkhead when not in use will be provided. A reinforced concrete flood wall, starting at the north line of Loring Street, will extend northeasterly along the top of the valley bank and the east side of Shawsheen Road to the south right-of-way line of the Boston and Maine Railroad and then extends westerly along the north line of Portland Street ending near Osgood Street. The wall will be reinforced concrete designed in the form of a modified T section which will vary in section with the height of the wall required to make the uniform top elevation 41.0. It is planned to install drain lines to carry off surface drainage and seepage throughout the length of the wall and dike, excepting the Portland Street section. The presence of an existing street drain along Portland Street makes the construction of additional drainage facilities unnecessary.

(3) West Lawrence.- The W.P.A. project initiated in 1940 planned for 2300 feet of earth dike and about 725 feet of concrete walls. Approximately 1000 feet of the dike were finished according to the plans. The remaining 1300 feet is in various stages of completion. Work was not started on the proposed section of the concrete wall. For the partially completed 1300 feet section of earth dike, it will be necessary to provide riprap for the protection of the river side of the dike; to install an extension of a toe drain on the land side for intercepting seepage flow; and to place topsoil and seed the dike. Some additional fill and regrading will be required for sections of the dike where the impervious fill had been placed but not shaped into the final cross section. A concrete wall will be constructed along the river bank at the O'Sullivan Park in order not to restrict the use of the Park and also for the more favorable appearance of the wall compared with the earth dike. The wall will be a modified T section similar to

the proposed design for North Andover and south Lawrence. The dumped and fill material will be excavated in the wall foundation area and replaced with a compacted impervious material. The test borings indicate that the depth of this fill material averages between 8 to 12 feet. A seepage trench consisting of an open joint vitrified tile drain in a trench backfilled with a pervious gravel will be provided for intercepting subsurface flows. East of the Park a shallow dike will be constructed similar to the W.P.A. dike. A minimum of 5 feet of fill will be removed and backfilled with impervious material. A waste pile of discarded city-owned cobblestones will be available for riprapping purposes. Similar use was made by the W.P.A. in the completed portion of the earth dike, and except for small areas that have been damaged wantonly, the cobblestone riprap appears to be satisfactory. It is proposed to repair the existing riprap in various sections that have been damaged.

d. Drainage and Pumping.-

(1) North Andover.- The existing drains and sewers, and proposed new drains, will be piped to a central pumping station, as shown on Plate 5. The existing 24-inch drainage outlet discharging into the Shawsheen River at the Massachusetts Avenue bridge will be plugged and abandoned. All storm drainage will be diverted into a new 36-inch drain and, normally, this flow will pass through the pumping station and discharge into the Shawsheen River. The site of the pumping station is selected to take advantage of the line of the existing outfall sewer and the necessity of combining the existing drainage system with the proposed drains to provide a single outlet. The sewage and drainage flows will be diverted into the suction conduit of the pumping station during flood stages, and pumped into a 36-inch outlet to the river. Consideration and study have been given to the characteristics and cost of various types of pumps in order to satisfy the pumping requirements outlined in Paragraph 1 (g) of Appendix "A". It is desirable to have pumps with a wide range of operating flexibility to meet the anticipated variable inflows from sewage and drainage which may range from 1 to 36 c.f.s.. As a result of this study it is considered essential to provide three fixed blade propeller pumps with the following maximum design capacities:

1 - 4 c.f.s. pump with an electric motor
2 - 23 c.f.s. pumps with gasoline motors.

These three pumps will have a total pumping capacity of 50 c.f.s., or 40 percent in excess of the discharge capacity of the sewers. If one large pump failed, the remaining two pumps would have a capacity of 27 c.f.s., or 75 percent of the maximum inflow. The 4 c.f.s. pump with electric motor will be of fixed capacity dependent only on the pumping head. The capacity of the two large pumps with gasoline motors is subject to considerable flexibility by operating the motors at variable speeds. This proposed pro-

cedure will undoubtedly sacrifice some of the pump efficiency, but, because of the infrequent use of the pumps, it is believed that flexibility of operation to satisfy the many possible conditions of sewage and storm run-off is a more essential criteria than pump efficiency and economy of operation. Consideration has been given to the possibility of using adjustable blade propeller pumps to obtain the desired range in pumping capacity, but it was concluded that, due to greater cost, and the additional mechanism to be maintained in operating condition by the municipalities, this type of pump was not warranted.

(2) South Lawrence.- The existing combination city sewers and drains and the proposed new lines all extend to the pumping station at the corner of Osgood and Market Streets as shown on Plate 5. The purpose of the new sewers is twofold:

(a) Provide an outlet for the surface run-off and seepage intercepted by the dike and wall, and

(b) Separate the entire discharge in Area "C" from the flow in the 54-inch trunk sewer.

This regulation and division of the flows will be obtained by an underground control chamber adjacent to the pumping station. During river stages below elevation 29.0, flow from Area "C" will pass through the control chamber directly into the 54-inch trunk main. When protection is required for Area "C" during flood stages (see paragraph 2 h. in Appendix "A"), a gate will be closed to prevent discharge into the 54-inch sewer, and the flow will be diverted through the proposed 36-inch pipe to the pumping station. The pumping station discharge will be carried in a 36-inch conduit to the 54-inch trunk main. When protection is necessary for both Areas "C" and "D" during higher flood stages, a gate located on the 54-inch main in the control chamber will be closed, the gate between the two sections of the chamber will be opened and the discharge from all inflow pipes will be diverted through the 36-inch pipe to the suction conduit in the pumping station. The discharge from the pumping station will be carried through the 36-inch discharge conduit back to the 54-inch main. Computations show that with all three pumps operating during high river stages, there is a drop of one foot in the hydraulic gradient due to frictional losses from the pumping station to the Merrimack River. If the flood stage reached elevation 40.0 (1 foot below top of the wall) water level at the pumping station would be about elevation 41.0. Due to the drop in gradient, it will be necessary to reconstruct the top of three existing manholes to make them watertight under the head created by backwater and pumping. The site selected for

the pumping station is a vacant lot located advantageously to the existing and proposed sewers. The station will have identical equipment as the North Andover station namely: three fixed blade propeller pumps with the following design capacities:

1 - 4 c.f.s. pump with an electric motor
2 - 23 c.f.s. pumps with gasoline motors.

Similar to the discussion of the North Andover pump, it is concluded that sufficient flexibility of operation can be obtained by varying the speeds of the gasoline motors.

e. Discussion.-

(1) Alternate Protective Plans Considered.- Two other schemes for the protection of these areas have been considered but neither have appeared to be economical. Brief descriptions of the plans considered are outlined in the following paragraphs:

(a) Construct an earth dam across the Shawsheen River just upstream from the Boston and Maine Railroad bridge. On the aerial photograph (Plate 3) this site would be approximately a straight line extension of the dike along the side of Portland Street. Gated conduits would permit ordinary discharges to flow normally to the Merrimack River. During flood stages the gates would be closed and the flow in the Shawsheen River would be pumped against the flood stage of the Merrimack River. As it is possible that the flood flow on the Shawsheen River would synchronize with the flood crest on the Merrimack River, it would be necessary to provide pumping capacity of approximately 1500 c.f.s.. The cost of the pumping station and equipment alone exceeds the entire cost of the recommended project. In addition to this large pumping plant, it would be necessary also to have pumping stations to (1) dispose of the sewage from the North Andover section, and (2) prevent backwater and flooding in the Lawrence section from the trunk sewer outlet.

(b) The second plan, to eliminate the cost of pumping the flow in the Shawsheen River, was the construction of two dams across the Shawsheen, one located as described in the preceding plan, and the second located upstream about opposite Elmwood Avenue in North Andover. These two dams would be connected by a conduit sufficient in size to carry the flood discharges of the Shawsheen River. A small pumping plant would be required to dispose of the local sewage and drainage. The cost of this plan greatly exceeds the proposed system of dikes and walls.

(2) Pump Capacity - Lawrence.- The capacity of the pumps for the Lawrence pumping station has been given careful con-

sideration. The proposed pumps will have a total capacity of approximately one-half the discharge capacity of the combination sewer. To provide pumps and stand-by units for the capacity of the sewer would increase the cost of the Lawrence project by approximately $75,000. It is considered that the benefits obtained by this additional protection do not justify the insurance. The chances are extremely remote that a storm of high intensity will occur simultaneously with high flood stages. In terms of probability (Plate 10), it would require a storm with a magnitude to be expected once in 10 years occurring concurrently with a flood stage to be expected once in one hundred years. If a storm which produced run-off in excess of the pumping capacity did take place during flood stages, the result would be surface storage in low areas and a few flooded cellars from backwater in the sewers. The duration of these conditions would be short without serious inconvenience to the affected areas. It is concluded that the factor of safety usually required for specifying pump capacity is not justified for this project.

7. Cost Estimate.- The estimated costs of the local protection projects for flood control at North Andover and south and west Lawrence are as follows:

a. North Andover.-

Wall and dike	$185,800
Pumping Station and drains	72,500
Channel realignment	5,400
Subtotal.....	$263,700
Engineering, Inspection, Overhead and Contingencies (25%)....	65,900
Total construction costs.....	$329,600

b. South Lawrence.-

Wall and dike	$136,200
Pumping Station and drains	90,400
Subtotal.....	$226,600
Engineering, Inspection, Overhead and Contingencies (25%)....	56,700
Total construction costs.....	$283,300

c. West Lawrence.-

Wall and dike	$ 41,800
Engineering, Inspection, Overhead and Contingencies (25%)....	10,450
Total construction costs.....	$ 52,250

d. Total Estimated Cost:-(3 sections) $665,150.
(For itemized analysis of cost estimates see Appendices B, C and D.)

8. Benefits of Projects.-

a. Methods of Analysis.- Reference is made to Section "B", "Data for Economic Justification", in "Appendix to Accompany Survey Report for Navigation, Flood Control and Water Power, Merrimack River, Massachusetts and New Hampshire" submitted by this office, dated 1 April 1940. Section "B" describes in detail the methods used in the economic analysis of flood control reservoirs based on direct and indirect flood losses. The same method applies to the economic analysis of local protection projects.

b. Determination of Benefits.- The proposed local flood protection works will supplement the effectiveness of flood protection provided at the damage centers by the upstream reservoirs and will increase the total benefits accruing to the comprehensive flood control program. The additional benefits resulting from the local protection projects have been evaluated and totalled with the benefits secured from the balance of the comprehensive flood control program to determine the economic justification for the proposed work. The proposed dikes and walls have been found to provide the most economical means of supplemental flood protection for North Andover and Lawrence. In addition to providing protection from floods, it is believed that the proposed project will be very beneficial to the general welfare of the community by promoting development of the low areas and enhancing property values. The costs and benefits involved in the comprehensive flood control program are summarized in the following tabulation:

Summary of Benefits and Costs

(1) Construction Costs:

(a) Local protection projects		
North Andover	$329,600.	
South Lawrence	283,300.	
West Lawrence	52,250.	
Subtotal		$ 665,150.
Other projects *		677,780.
(b) Flood Control Reservoirs **		16,987,000.
Total Construction Costs		$18,329,930.

(2) Annual Carrying Charges:

(a) Local protection projects		
North Andover	$ 14,070.	
South Lawrence	12,070.	
West Lawrence	2,290.	
Subtotal		$ 28,430.
Other projects *		36,460.
(b) Flood Control Reservoirs **		788,970.
Total Annual Carrying Charges		$ 853,860.

(3) Total Annual Benefits:

Based on comprehensive flood control program, including reservoirs and local protection..$1,060,500.

(4) Ratio of Annual Benefits to Annual Carrying Charges: 1.24

* Includes completed local protection at Lowell, Massachusetts and proposed project at Nashua, New Hampshire.

** Includes completed reservoirs at Franklin Falls and Blackwater, and proposed reservoirs at Bennington, Beards Brook, West Peterboro, and Mountain Brook.

9. Local Cooperation.-

a. Views of Local Interests.- The proposed projects have been discussed in a joint meeting on 9 March 1944 with the officials of North Andover and Lawrence in order to determine the attitude and local interest of the affected communities. The Lawrence projects were previously initiated and sponsored by the City as a W.P.A. project and the proposition that the flood control protection measures might be completed during a post-war construction program was favored by the Mayor and the City Council. The contemplated W.P.A. flood control projects in Lawrence in 1939 caused considerable agitation in North Andover for similar projects for equal protection. Consequently, the Board of Selectmen of North Andover indicated their interest and desire for the proposed project. Both municipalities expressed the urgency for flood protection to eliminate further repetition of flood damages that have occurred during the past 10 years.

b. Extent of Local Cooperation Required.- In accordance with Section 3 of the Flood Control Act, approved 22 June 1936, the local municipalities will be required to furnish all lands, easements, and rights-of-way necessary for the construction of the project. The estimated costs to local interests for these items is $10,000 for North Andover, $5000 for south Lawrence and $4000 for west Lawrence.

c. Assurance of Local Cooperation.- The City of Lawrence, by official action of the City Council and the Mayor on 13 March 1944, and the Town of North Andover in a special town meeting held 27 March 1944, have furnished the necessary assurance that the municipalities will (1) furnish without cost to the United States, all lands, easements, and rights-of-way necessary for the project; (2) hold and save the United States free from all claims for damages due to the work; and (3) maintain and operate the project without expense to the United States, in accordance with regulations prescribed by the Secretary of War. Photostat copies of the resolutions adopted by Lawrence and North Andover are attached as Appendices "F" and "G".

10. Time Required for Construction.- It is estimated that the projects proposed herein can be completed in one construction season.

11. Recommendation.- It is recommended that the local flood protection works proposed herein, at an estimated cost to the United States of $665,150 be selected as a definite project to supplement the comprehensive plan for reservoirs and related flood control works for the Merrimack River Basin as authorized by the Flood Control Acts of 1936 and 1938.

SUPERSEDED

H. P. DUNBAR
Lt. Col., Corps of Engineers
District Engineer

APPENDIX "A"

HYDROLOGY OF THE DRAINAGE AREAS

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APPENDIX "A"

HYDROLOGY OF THE DRAINAGE BASINS.

1. North Andover.-

a. Drainage Area.- The total area that normally drains into the Shawsheen River through the section to be diked consists of 78 acres, approximately 2/3 residential development and the remainder consisting of a gravel pit and farmland. The land slopes uniformly from approximately elevation 80 at Railroad Avenue (Plate 8) to approximately elevation 30 along the top of the slopes overlooking the Shawsheen River. The drainage area is divided into two sub-areas "A" and "B" for purposes of analysis. Area "A" (13 acres) is the section bordered by Massachusetts Avenue and the proposed flood wall south of the Avenue. Most of the present surface drainage in this area is overland to the Shawsheen River without any underground drainage system. It will be necessary to install a drainage system for this area as shown on Plate 5 to carry off the storm water that will be intercepted by the dike and wall. Sub-drainage also will be provided. Area "B" (65 acres) is the remainder of the drainage area lying north of Massachusetts Avenue, and except for the gravel pit and cultivated sections, the area has a drainage system of catchbasins and drains that combine and ultimately discharge under the Massachusetts Avenue bridge into the Shawsheen River. There is no defined drainage pattern to the area and no brooks or drain valleys to accumulate the run-off. The drains vary in size from 8" to 24", with a total computed capacity of about 20 c.f.s.. Any future development of the farm and gravel pit sections in Area "B" with corresponding extensions of the drainage system will not necessarily augment the pumping requirements because the surface run-off can be discharged directly into the Shawsheen River.

b. Sewerage.- The entire section (Areas "A" and "B"), with the exception of a few isolated residences, has a sanitary sewerage system to separate the sewage from the surface run-off. A 24-inch trunk sewer along the east side of the Shawsheen River intercepts the street sewers and carries the sewage to the Merrimack River. The 24-inch trunk sewer extends south on Massachusetts Avenue only as far as Beechwood Street. If North Andover developes sufficiently, the trunk sewer will be extended west on Beechwood Street to the Shawsheen River and then south along the east bank of the river. Approximately 1.5 square miles of North Andover would be served by this sewer if such an extension were required. The capacity of the existing 24-inch sewer is 6 c.f.s. which is adequate for the ultimate scheme, and is more than required for the present development of the town.

c. Storm Rainfall.- The rainfall intensity data prepared by David L. Yarnell and published in the United States Department of Agriculture Publication No. 204 were used for this study since there are no rainfall records for Lawrence or North Andover suitable for use as maximum precipitation rates for storms of comparatively short duration. The hydrological analyses of these two sites are based on rainfall rates for 15, 30 and 60 minute durations for various frequencies in years. The time of concentration was obtained to determine the required duration of rainfall intensity to produce the maximum discharge. The time of concentration for all areas was computed by assuming an average velocity of flow from the most distant part of the drainage area to the pumping station and by measuring the distances along either existing or proposed drain lines. The computed time of concentration for Area "A" is 15 minutes and for Area "B" the time is 30 minutes. Consequently, 15 minute rainfall intensities were used for determining the required size of drains in Area "A" and 30 minute intensities were used for determining theoretical pumping capacity from the total area. The maximum rainfall in inches per hour for various frequencies are tabulated below:

Frequency In Years	15 Minute Duration	30 Minute Duration
2	2.8	1.8
5	3.6	2.4
10	4.1	2.8
25	5.0	3.8

d. Storm Run-off.- The storm run-off was based on the Rational Formula $Q = C\ I\ A$, in which

Q = Run-off in c.f.s.

C = Coefficient, dependent on the soil conditions and characteristics of the topography, expressing the ratio of the rate of run-off to the rate of rainfall.

I = The maximum average intensity of rainfall over the entire drainage area, in inches per hour, which may occur during the time of concentration.

A = Drainage area in acres.

Consideration was given to other accepted methods of computing surface run-off such as the Burkli-Ziegler Formula and the synthetic unit hydrograph, but due to the nature and size of the

drainage area and the lack of basic data, it was concluded the Rational Formula was the most applicable. The values of the coefficient "C" were estimated from an evaluation of the types, slopes, and relative imperviousness of the different surfaces, and were estimated as follows:

(1) Paved areas, including streets, sidewalks, roofs, etc..... 0.35

(2) Lawns, gardens, and undeveloped areas..... 0.15

(3) Gravel pit and adjacent drainage area..... 0.05

The average rates of run-off applicable to Areas "A" and "B" were determined as follows:

Area "A".-

Paved area -	3.5 acres	x 0.35 =	1.225
Lawns, gardens, etc.	9.5 acres	x 0.15 =	1.425
Total.....	13.0 acres		2.650

Average "C" (rate of run-off) = $\frac{2.650}{13.0} = 0.20$

Area "B".-

Paved area -	20 acres	x 0.35 =	7.00
Lawns, gardens, etc.	35 acres	x 0.15 =	5.25
Gravel Pit, etc.	10 acres	x 0.05 =	0.50
Total.....	65 acres		12.75

Average "C" (rate of run-off) = $\frac{12.75}{65} = 0.20$

The theoretical amount of storm run-off to be provided for was computed from the preceding data as follows:

Area "A" (15 Minute Duration)		Area "B" (30 Minute Duration)	
Frequency In Years	Rainfall Rate In In./ Hr.	Storm Run-off c.f.s.	Rainfall Rate In In./ Hr.	Storm Run-off c.f.s.
2	2.8	7.3	1.8	23.4
5	3.6	9.4	2.4	31.2
10	4.1	10.7	2.8	36.4
25	5.0	13.0	3.8	49.4

Similar computations made on the basis of 30 minute concentration and a 10 year frequency of the total 78 acres to determine the theoretical pumping capacity gave a storm run-off rate of 44 c.f.s. and a total rate, including sewage and seepage, of 51 c.f.s..

e. Seepage.- A seepage allowance of .0001 c.f.s. per linear foot per 10 feet of head is provided as a result of the sub-surface explorations at the site and an analysis of the material encountered. This allowance results in a total seepage flow of about 1/4 c.f.s. at North Andover.

f. Capacity of Proposed Drains.- It is proposed to design the drainage system for Area "A" for a storm of a ten year frequency although the capacity of the existing drainage system serving Area "B" theoretically is adequate for a storm of only a 2 year frequency. It is believed essential to provide for the 10 year frequency for Area "A" because of the nearness of the flood wall to the residences. Surface storage that might be created from a storm exceeding the capacity of the drains would flood cellars and result in some property damage. Excess run-off from Area "B", not intercepted by the catchbasins and drain in Massachusetts Avenue, also would tend to flow overland until checked by the flood wall. Drain capacities are computed based on the criteria that the drains are flowing full, and using a value of "n" in Kutter's formula equal to 0.015. The layout of the proposed drainage system for Area "A", its combination with the drainage from Area "B", the pumping station, and discharge to the Shawsheen River, is shown on Plate 5.

g. Pumping Requirements.- The total inflow capacity to the pumping station, including the existing sewerage and drainage pipes and the proposed new drains is approximately 36 c.f.s.. The range in the inflows that may be anticipated vary from the normal sewage discharge of less than 1 c.f.s. to this maximum flow with storm run-off of 36 c.f.s.. The pumping head, produced by the river flood stages also is a variable depending on the magnitude of the flood at the time of some local storm. It is believed essential to provide a pump of small capacity, or one with sufficient range to pump small flows, for it is expected that when pumping is required, the probability is that the flood stages will occur during the clear weather that normally follows a flood producing storm and that there would be no drainage run-off. Although it will be very infrequent that storms of severe intensity will occur simultaneously with the high river stages (see Paragraph 4 b.) the pumps will also have a capacity based on the assumption that a rainfall, with an intensity to be expected once in 10 years, will take place during high river stages. With this severe criteria, it was concluded that the pumps should have a 33% surplus capacity for the existing conditions, and approximately equal to the flow from an ultimate development, assuming drainage

diversion did not materialize during the future growth of the area. To satisfy these conditions, it is proposed to utilize three fixed blade propeller pumps of the following capacities:

1 - 4 c.f.s. pump with an electric motor
2 - 23 c.f.s. pumps with gasoline motors.

Further discussion concerning selection of pumps is included in paragraph 6 d. (1) of the report.

2. Lawrence.-

a. Drainage Area.- The hydrologic study of the drainage area is governed by the existing sewerage and drainage systems which necessitate considering separate sections of the area individually. A 54-inch diameter brick sewer in Osgood Street serves as a trunk outlet for both sanitary sewage and storm drainage in Areas "C" and "D", and for sanitary sewage from Area "E" (Plate 8). Descriptions of the different areas are as follows:

(1) Area "C" consists of approximately 40 acres bounded by the proposed flood wall and Osgood Street. This area comprises the major portion flooded by past floods, and is the area obtaining the direct benefit of the flood wall. This area also is differentiated because of the sewerage and drainage problems that arise and are discussed further in paragraph 6 d. (2) of the report. Existing combination sanitary and storm sewers discharging into the 54-inch trunk main in Osgood Street are indicated on Plate 5.

(2) Area "D" consists of 160 acres and is analyzed separately because most of the area is at elevations not subject to river flooding. Pumping would be required for this area only during extremely high flood stages. Sewage and storm drainage are combined in the same system.

(3) Area "E" consists of approximately 104 acres that is above all flood levels. Only sanitary sewage discharges from this area into the trunk mains in Area "D" which eventually discharge through the 54-inch outlet to the Merrimack River. Storm run-off is intercepted by a separate system and is discharged into natural stream beds that flow into the Shawsheen River. All areas, except O'Connell Park are fully developed, as shown by the aerial photographs, Plates 2 and 3. The areas are primarily residential with some commercial establishments along the main streets in Areas "D" and "E". Surface drainage is intercepted in all the roads by a system of catchbasins and discharges

into the combination sanitary and storm sewer. A large trunk main, increasing from 24 to 54 inches, located as shown on Plate 8, intercepts the lateral drains and discharges into the Merrimack River.

b. Storm Rainfall.- The rainfall data used for the North Andover analysis and described in Paragraph 1 c. of this Appendix is applicable to the Lawrence project. Two separate analyses were made in order to select sizes of new drains and capacity requirements for the pumps. These two analyses were the determination of the maximum run-off from Area "C", and from Areas "C" and "D" combined. The time of concentration of the surface run-off from the 40 acres of Area "C" was computed to be approximately 30 minutes, and from the 200 acres comprising both Areas "C" and "D", the time of concentration was computed to be about 1 hour.

c. Storm Run-off.- The rational formula, $Q = C\ I\ A$, was used to determine the maximum rate of run-off. The computed average value of the run-off coefficient was 0.28 for Area "C". It was impossible to divide and segregate the drainage area of the combined Areas "C" and "D" into subareas of different surface characteristics, so the average run-off coefficient 'C' was estimated to be 0.30. The theoretical amount of storm run-off was computed from this data, as follows:

	Area "C" (30 Minute Duration)		Areas "C" and "D" (1 Hour Duration)	
Frequency In Years	Rainfall Rate Inches/Hr.	Storm Run-off c.f.s.	Rainfall Rate Inches/Hr.	Storm Run-off c.f.s.
2	1.8	20	1.1	66
5	2.4	27	1.5	90
10	2.8	31	1.8	108
25	3.8	43	2.3	138

d. Sewage Flow.- The maximum rate of sewage flow from the entire area is based on the following assumptions:

Population = 30 persons per acre.

Daily Discharge = 100 gallons per capita.

Load Factor (ratio of maximum to average flow) = 3.

The average flow, based on these assumptions, was computed to be 1.4 c.f.s. with a maximum rate of flow of about 4.2 c.f.s..

e. Seepage.- Seepage under the concrete wall and through the earth dike is estimated to be 0.5 c.f.s..

f. Capacity of Existing Sewers.- The capacities of the existing sewers computed on the basis of their size and slope, and a value of "n" in Kutter's formula of 0.015 are as follows:

	Size	Capacity c.f.s.
Portland Street	15"	3.5
Market Street	15"	2.7
Greenfield Street	10"	1.0
Salem Street	15"	3.5
Existing total capacity of sewers serving Area "C" and discharging into the 54" main in Osgood Street		10.7
Salem and Osgood Streets (based on average slope between South Union and Portland Streets)	54"	100

g. Proposed Sanitary and Storm Sewers.- The general plan of the proposed sanitary and storm sewers is shown on Plate 5. Surface run-off that will be intercepted by the wall and dike will be disposed of by discharging into the existing sewer from new catchbasins and laterals, and by new drainage lines extending along the wall and dike. The new drains will slope to Loring Street, intercept two existing drains that discharge into the Shawsheen River, and then extend along Loring Street, Salem Street and Osgood Street to a proposed control chamber located at the Corner of Osgood Street and Market Street. The direction of flow is opposite to the surface slope and the depth of the drains therefore, will increase from 8 feet at Loring and Salem Streets to 20 feet in Osgood Street. This same line will also intercept the flow from the existing sewers in Salem, Greenfield, Market and Portland Streets in order to be able to isolate the entire sewage and drainage in Area "C" from the flow in the 54 inch main. The purpose of this layout is described further in paragraph 6 d. (2) of the report. The capacity of the proposed 15 inch drain in Salem Street will be 3.5 c.f.s. which will increase the total sewer and drain capacity from Area "C" to 14.2 c.f.s.. If the storm to be expected once in 10 years is adopted as the design criteria, this total capacity is low compared with the theoretical computed 31 c.f.s.. The City Engineer, Lawrence, has advised that no difficulty has been encountered from these combination sewers and drains, hence it is concluded that the system can be utilized safely without increasing the size of the system. The capacity of the 54 inch sewer closely checks the computed run-off and is believed to be satisfactory.

h. Pumping Requirements.- The pumping requirements are

influenced by the frequency that pumping will be required by different heights of river levels. A curve showing the expected relation between river stages and frequency is shown on Plate 10. Assuming that pumping is necessary when the river stage rises to an elevation approximately 6 feet below the lowest average ground level to prevent sewage from backing into cellars, pumping for protection of Area "C" should commence with river stage at about elevation 29.0. Similarly, pumping for Area "D" should start at a river stage of approximately 34.0. The frequencies that the river is anticipated to reach these stages are once in 12 years to elevation 29.0 and once in 110 years to elevation 34.0. Data have been compiled from past records and are plotted on Plate 11 to show the 1 hour rainfall intensities that have occurred simultaneously with high river stages. This curve illustrates the improbability of high intensity rainfall occurring during flood stages. The rainfall of high intensity usually takes place during the storm that produces the flood, as noted in paragraph 4 b, of the report. Flood stages at Lawrence occur after more than 2 days following the storm and ordinarily the center of the meteoroligical disturbance has passed by that time. Consequently, the selected design criteria for determining the capacity of the proposed pumps are as follows:

(1) Condition No. 1.-
- (a) River stage elevation 38.8 (Design storm).
- (b) Rainfall is negligible.
- (c) Pump required for maximum sewage discharge from Areas "C", "D" and "E". Maximum computed discharge is 4 c.f.s..

(2) Condition No. 2.-
- (a) River stage between elevations 29.0 and 34.0.
- (b) Frequency of this occurrence is once in 12 years.
- (c) Rainfall of an intensity to be expected once in 10 years occurring simultaneously with the high river stage.
- (d) Pumping required for protection of Area "C". Maximum computed discharge, including sewage, is 33 c.f.s.. Capacity of sewers, existing and proposed, is 14.2 c.f.s.. Discharge from Areas "D" and "E" can be allowed to flow to the Merrimack River by gravity without causing damage.

(3) Condition No. 3.-

(a) River stage elevations 34.0 and higher.

(b) Frequency of this occurrence once in 110 years.

(c) Rainfall of an assumed intensity of 0.5 inches per hour occurring simultaneously with the high river stage.

(d) Pumps required for pumping combined sewage and storm run-off from Areas "C" and "D" and sewage from Area "E". Computed discharge for this condition, including sewage, is 32 c.f.s..

It is proposed to provide pumps of the same type and capacity as those recommended for the North Andover project. Three fixed blade propeller pumps will have the following capacities against a maximum pumping head of approximately 25 feet:

1 - 4 c.f.s. pump with an electric motor
2 - 23 c.f.s. pumps with gasoline motors.

Further discussion concerning selection of pumps is included in paragraph 6 d. (1) of the report.

APPENDIX "B"

COST ESTIMATE

LOCAL PROTECTION, NORTH ANDOVER, MASS.

Item	Quantity	Unit Cost	Cost	
Wall and Dike				
Excavation	11,000 c.y.	$ 0.60	$ 6,600	
Stripping	4,000 c.y.	0.50	2,000	
Backfill	2,000 c.y.	0.40	800	
Compacted Impervious Fill	15,000 c.y.	1.25	18,800	
Compacted Fill	11,000 c.y.	1.25	13,800	
Sand and Gravel	5,000 c.y.	1.75	8,800	
Topsoil	3,000 c.y.	2.75	8,300	
Grading and Seeding	4.0 acres	375.00	1,500	
Riprap	500 s.y.	5.00	2,500	
Concrete	4,260 c.y.	25.00	106,500	
Reinforcing Steel	124,000 lbs.	0.07	8,700	
Bulkhead		L.S.	7,500	
Cost of Wall and Dike				$185,800
Drains and Pumping Station				
Drains		L.S.	12,500	
Pumping Station		L.S.	60,000	
Cost of Drains, Pumping Station				72,500
Channel Realignment	12,000 c.y.	0.45		5,400
Subtotal				$263,700
Engineering, Inspection, Overhead and Contingencies (25%)				65,900
Total Construction Costs - North Andover				$329,600

APPENDIX "C"

COST ESTIMATE

LOCAL PROTECTION - SOUTH LAWRENCE, MASS.

Item	Quantity	Unit Cost	Cost	
Wall and Dike				
Excavation	13,700 c.y.	$ 0.60	$ 8,200	
Stripping	600 c.y.	0.50	300	
Backfill	3,500 c.y.	0.40	1,400	
Compacted Impervious Fill	10,500 c.y.	1.25	13,100	
Sand and Gravel	4,000 c.y.	1.75	7,000	
Topsoil	2,000 c.y.	2.75	5,500	
Grading and Seeding	2.5 acres	375.00	1,000	
Riprap	275 s.y.	5.00	1,400	
Concrete	3,400 c.y.	25.00	85,000	
Reinforcing Steel	83,500 lbs.	0.07	5,800	
Bulkhead		L.S.	7,500	
Cost of Wall and Dike				$136,200
Drains and Pumping Station				
Drains		L.S.	30,400	
Control Chamber		L.S.	5,000	
Pumping Station		L.S.	55,000	
Cost of Drains and Pumping Station				90,400
Subtotal				$226,600
Engineering, Inspection, Overhead and Contingencies (25%)				56,700
Total Construction Costs - south Lawrence				$283,300

SUPERSEDED

APPENDIX "E"

TABLE OF COSTS AND ANNUAL CHARGES

		North Andover	South Lawrence	West Lawrence
I.	Federal Investment			
	A. Structures (50 year life) Construction costs	$ 299,600.	$ 253,300.	$ 52,250.
	B. Equipment (30 year life)	30,000.	30,000.	none
	C. Total Federal Investment	$ 329,600.	$ 283,300.	$ 52,250.
	D. Annual Federal Carrying Charges			
	1. 3% Interest	$ 9,890.	$ 8,500.	$ 1,570.
	2. Amortization, structures (0.886%)	2,650.	2,240.	460.
	3. Amortization, equipment (2.102%)	630.	630.	-
	4. Total Federal Carrying Charges	$ 13,170.	$ 11,370.	$ 2,030.
II.	Non-Federal Investment			
	A. Land and Easements	$ 10,000.	$ 5,000.	$ 4,000.
	B. Annual Carrying Charges - 4% Interest	400.	200.	160.
	C. Annual Cost of Maintenance and Operation	500.	500.	100.
	D. Total Non-Federal Carrying Charges	$ 900.	$ 700.	$ 260.
III.	Total Annual Carrying Charges	$ 14,070.	$ 12,070.	$ 2,290.

SUPERSEDED

APPENDIX "H"

INDEX TO PLATES

Plate No.	Title
1	Flood Control Projects, Merrimack River Basin
2	Aerial Photograph, Lawrence and North Andover
3	Aerial Photograph, Lawrence and North Andover
4	Aerial Photograph, West Lawrence
5	Preliminary Layout, South Lawrence and North Andover
6	Cross-sections and Details, Lawrence and North Andover
7	Record of Exploration, Lawrence and North Andover
8	Drainage Areas, Lawrence and North Andover
9	Stage-Discharge Rating Curve, Lawrence and North Andover
10	Stage Frequency Curve, Lawrence and North Andover
11	Rainfall Intensity vs River Stage, Lawrence and North Andover
12	Preliminary Layout, West Lawrence
13	Stage-Discharge Rating Curve, West Lawrence.

MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION
LAWRENCE & NO. ANDOVER
AERIAL PHOTO
U. S. ENGINEER OFFICE, BOSTON, MASS., 1/44
0 800 1600
SCALE OF FEET

PLATE 8

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

MERRIMACK RIVER

Flood

MERRIMACK ST.

BOSTON AND MAINE RAILROAD

TO BOSTON

TO PORTLAND

PORTLAND ST.

TEMPLE ST.

MARKET ST.

HURST ST.

EAST

GREENFIELD ST.

SALEM ST.

LORING ST.

SHAWSHEEN RD.

OSGOOD ST.

EAILEY ST.

ABBOT ST.

MARBLEHEAD ST.

BEVERLY ST.

PERRY ST.

MIDDLESEX

MASSACHUSETTS AVENUE

COMMONWEALTH AVE.

TECHWOOD ST.

BRUCE ST.

DANVERS ST.

ELMWOOD ST.

TOWN OF NORTH ANDOVER

CITY OF LAWRENCE

SHAWSHEEN RIVER

New River Channel

Pumping Station

Control Chamber

Manhole "A"

Bulkhead Required

Approx. limit of dump

Top of slope

Toe of slope

Fill

Fill old channel

Earth Dike

Town of Andover outfall sewer

24" Sewer

1936 High Water Stage

Normal Water El = 14.5

Abandon existing outlets

DETAIL "B"

DETAIL "C"

DETAIL "D"

Note:- For proposed invert grades in Control Chamber & Pumping Station see PLATE 5

Note:- Manhole "A" located on the Andover outfall sewer, to be made water-tight for flood stage condition.

Note For Cross-sections and Details see PLATE 6

LEGEND

PROPOSED	EXISTING
DRAIN	SEWER
SEWER	DRAIN
UNDERDRAIN	
MANHOLE	MANHOLE
CATCHBASIN	CATCHBASIN
	AUGER BORINGS & TEST PIT
	DRILLHOLE

SUPERSEDED

MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION — SHAWSHEEN RIVER
LAWRENCE & NORTH ANDOVER, MASS.
PRELIMINARY LAYOUT

SCALE OF FEET 100 0 100 200 300 400

U. S. ENGINEER OFFICE, BOSTON, MASS. JANUARY 1944

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: LT. COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: CHIEF, DESIGN SECTION, BR. ENGINEER

Elliot F. Chiles, ENGINEER

DES. BY
DR. BY
TR. BY
CH. BY

REVISIONS

NO.	CHARACTER	DATE	BY

FILE NO. M55-12/20

PLATE 5

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

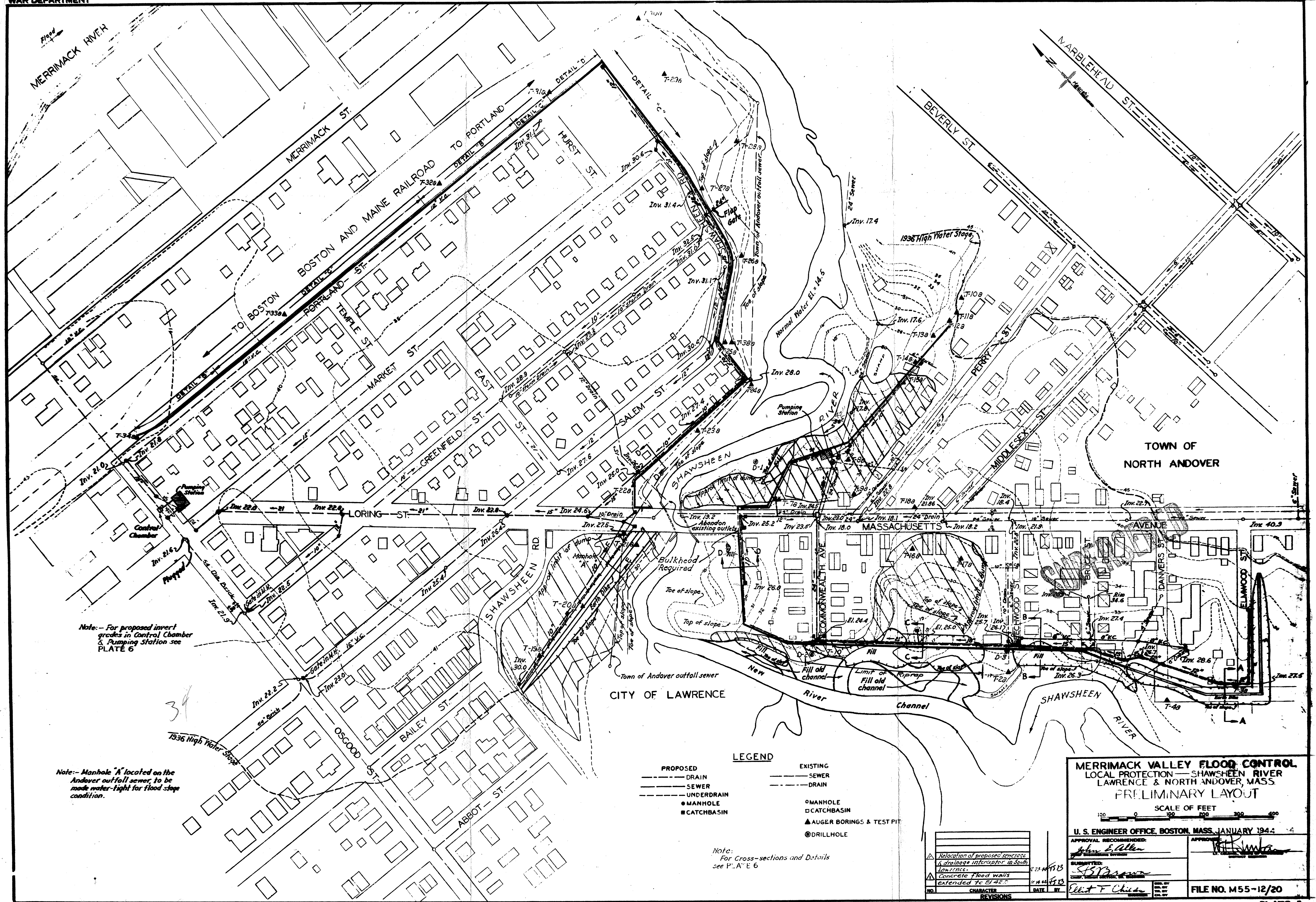
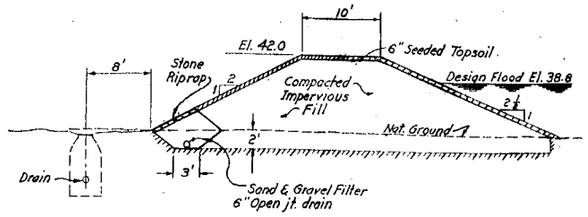


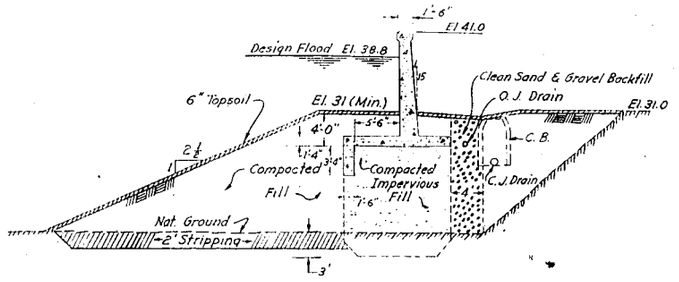
PLATE 5



SECTION A–A

TYPICAL CROSS SECTION

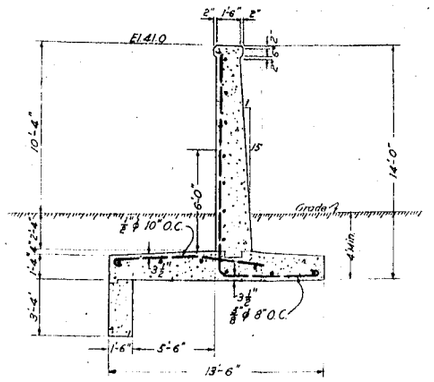
EARTH DIKE



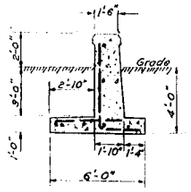
SECTION B–B

TYPICAL CROSS SECTION

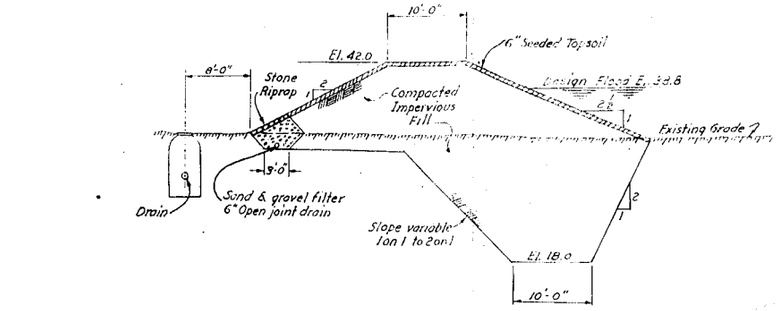
CONCRETE FLOOD WALL
ON PERVIOUS GROUND



DETAIL "A"



DETAIL "B"

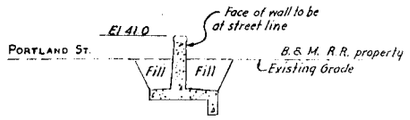


SECTION E–E

TYPICAL CROSS SECTION

EARTH DIKE

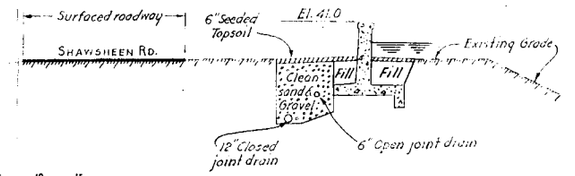
SOUTH LAWRENCE



TYPICAL CROSS SECTION

FLOOD WALL

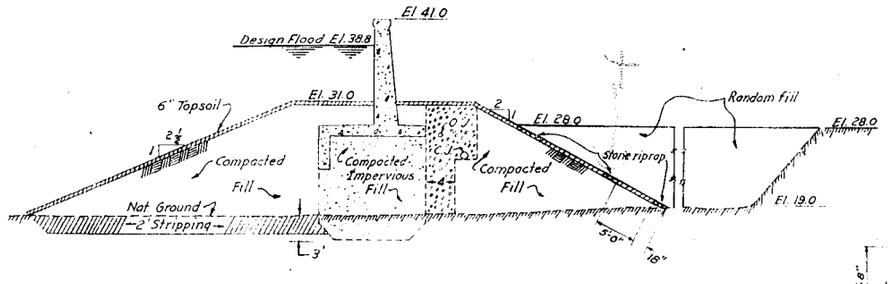
PORTLAND ST. SO. LAWRENCE.



TYPICAL CROSS SECTION

FLOOD WALL

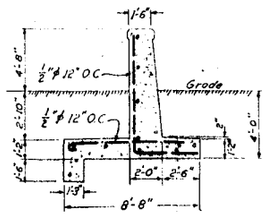
ALONG SHAWSHEEN RD. BETWEEN
SALEM ST. & GREENFIELD ST.



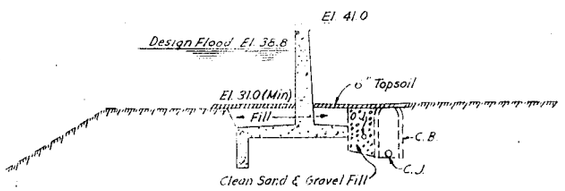
SECTION C–C

TYPICAL CROSS SECTION

CONCRETE FLOOD WALL ON FILL



DETAIL "C"



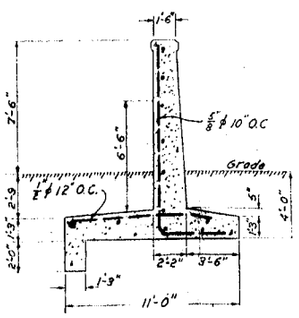
SECTION D–D

TYPICAL CROSS SECTION

CONCRETE FLOOD WALL
ON IMPERVIOUS GROUND

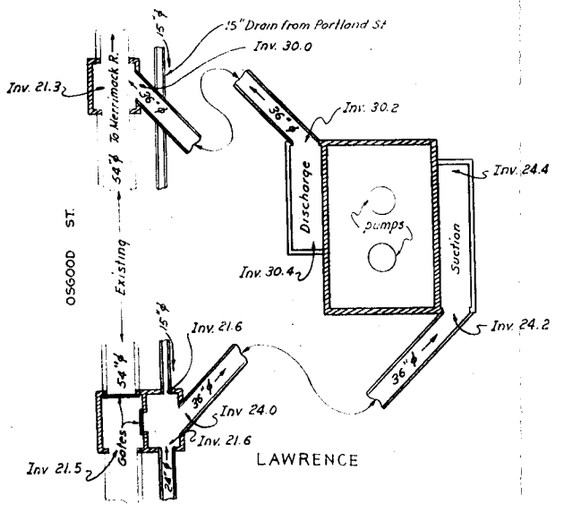
1.0 5 10 15
SCALE OF FEET

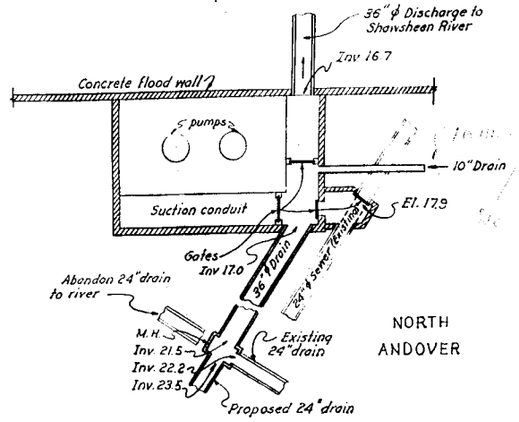
Note:– Indicated stage of the design flood is based on the March, 1936 flood, reduced by the comprehensive reservoir system. Earth dikes and concrete walls are structurally designed with flood stage at top elevation.



DETAIL "D"

1.0 5 10
SCALE OF FEET





PUMPING STATIONS
PRELIMINARY LAYOUT PLANS
NOT TO SCALE

NO.	CHARACTER	DATE	BY
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION—SHAWSHEEN RIVER
LAWRENCE & NORTH ANDOVER, MASS.

CROSS SECTIONS AND DETAILS

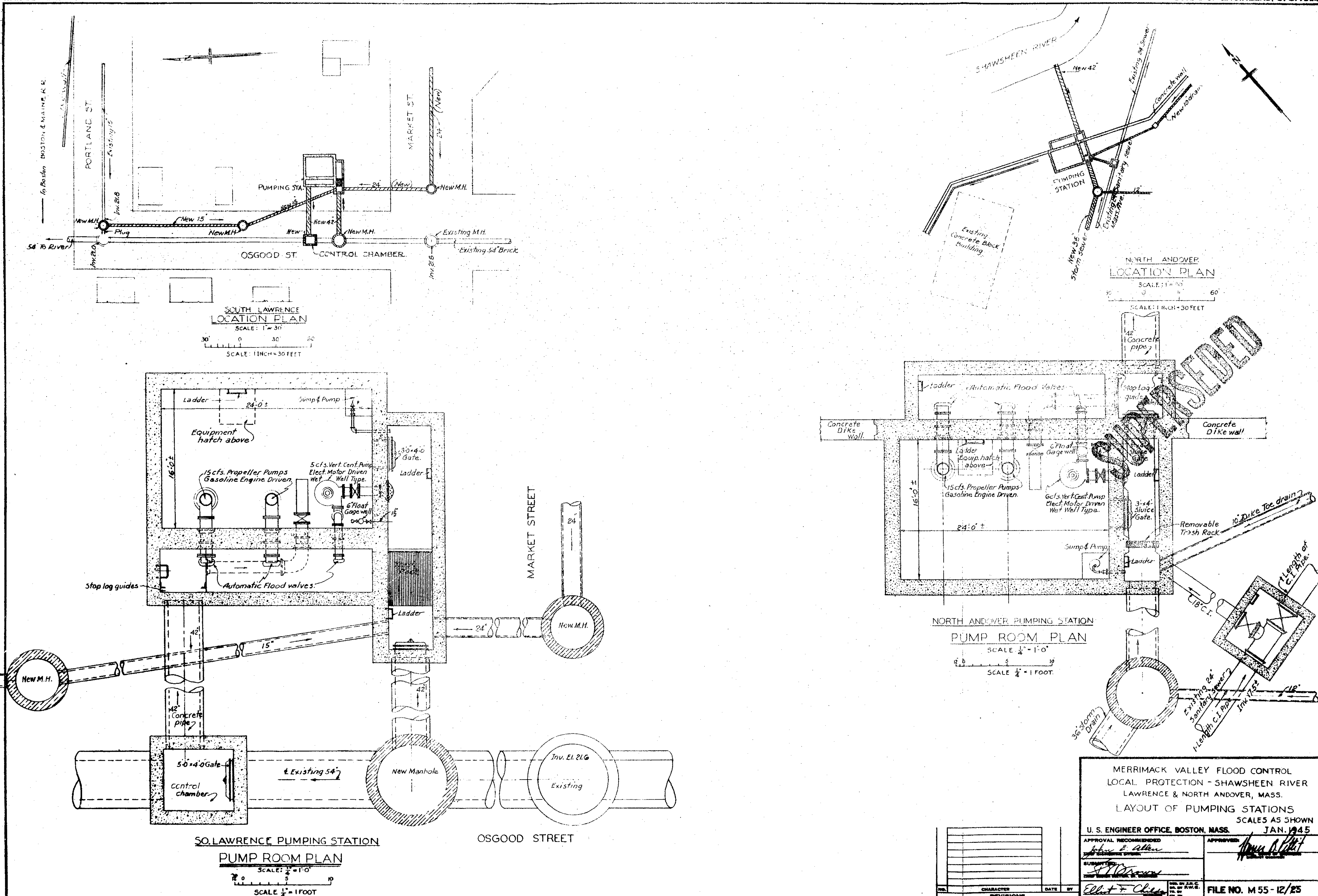
SCALES AS SHOWN

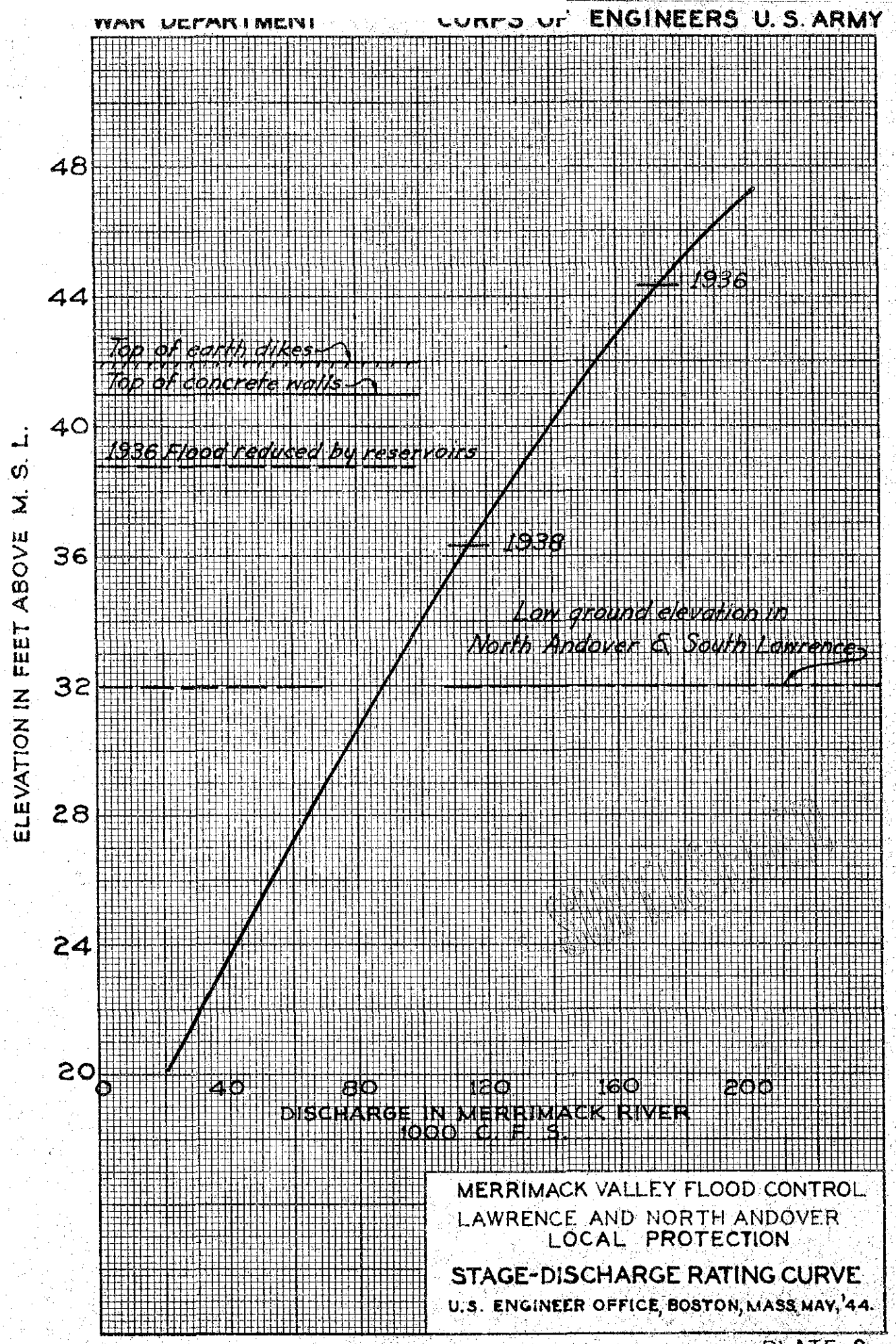
U. S. ENGINEER OFFICE, BOSTON, MASS. MAY, 1944.

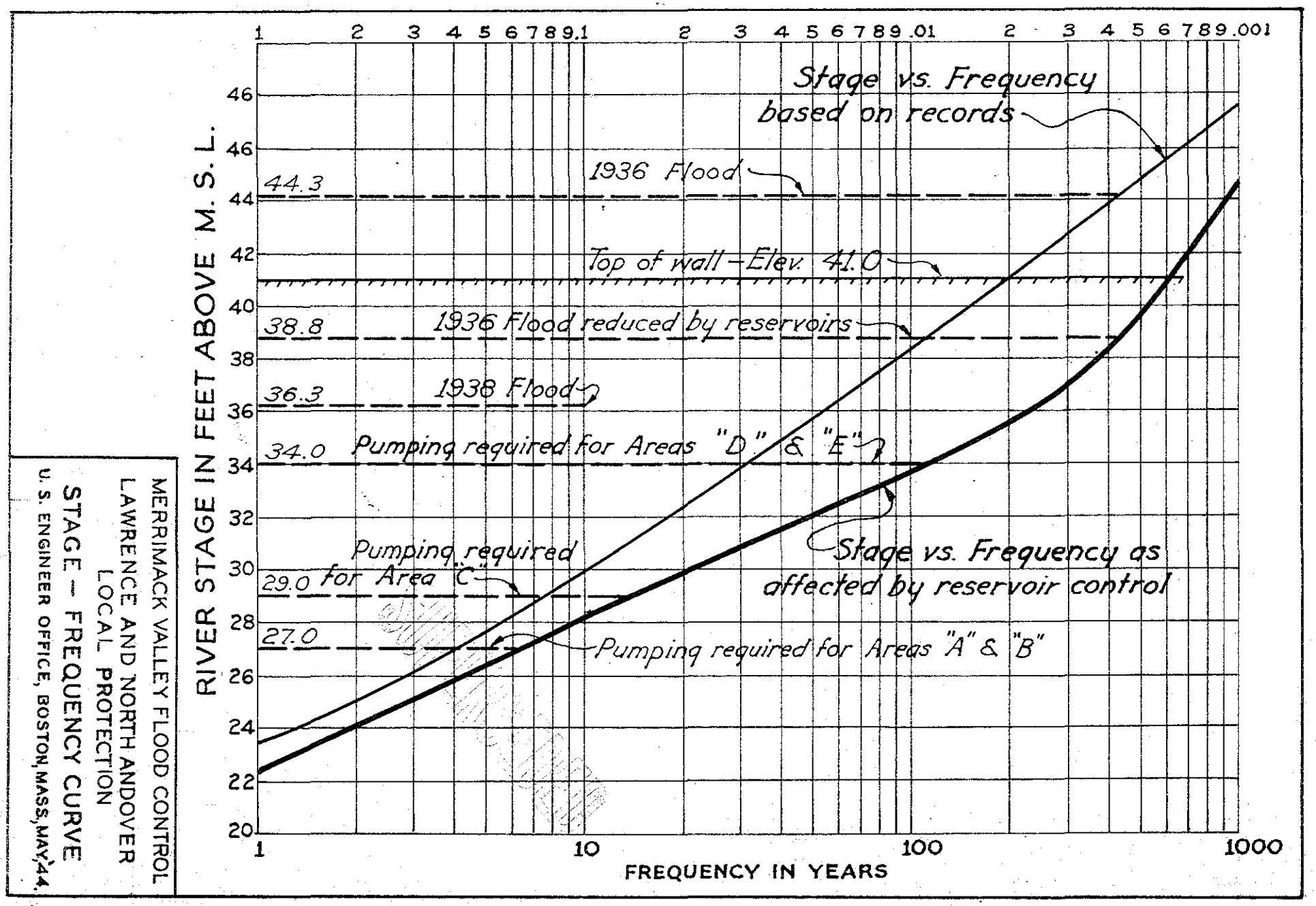
APPROVAL RECOMMENDED: John E. Allen
APPROVED: [illegible]
SUBMITTED: [illegible]
Elliot F. Chase
DES. BY
CH. BY
TR. BY
CH. BY

FILE NO. M55-12/21

PLATE 6


SOUTH LAWRENCE
LOCATION PLAN
SCALE: 1" = 30'
SCALE: 1 INCH = 30 FEET
To Boston BOSTON & MAINE R.R.
PORTLAND ST.
Existing 15"
MARKET ST.
24" (New)
PUMPING STA.
24" (New)
New M.H.
New M.H.
New 15"
New M.H.
New 15"
New 42"
New M.H.
Existing M.H.
Plug
54" To River
Inv. 21.8
Inv. 21.0
Inv. 21.6
OSGOOD ST.
CONTROL CHAMBER
Existing 54" Brick
SHAWSHEEN RIVER
New 42"
Existing 24" Sewer
Concrete wall
New 10" drain
PUMPING STATION
Existing Concrete Block Building
New 36" Storm Sewer
Existing Sanitary Sewer
Mass. Ave.
12"
NORTH ANDOVER
LOCATION PLAN
SCALE: 1" = 30'
SCALE: 1 INCH = 30 FEET
Ladder
24'-0"±
Sump & Pump
Equipment hatch above
16'-0"±
15 cfs. Propeller Pumps Gasoline Engine Driven
5 cfs. Vert. Cent. Pump Elect. Motor Driven Wet Well Type.
6" Float Gage well
30"x4'-0" Gate.
Ladder
Stop log guides
Automatic Flood valves.
Ladder
MARKET STREET
24"
New M.H.
24"
15"
42"
New M.H.
42"
42" Concrete pipe
5'-0"x4'-0" Gate
Control chamber
℄ Existing 54"
New Manhole
Inv. El. 21.6
Existing
SO. LAWRENCE PUMPING STATION
PUMP ROOM PLAN
SCALE: 1/4" = 1'-0"
SCALE 1/4" = 1 FOOT
OSGOOD STREET
42" Concrete pipe
Ladder
Automatic Flood Valves
Stop log guides
Concrete Dike Wall
Concrete Dike wall
Ladder
Equip. hatch above
6" Float Gage well
15 cfs. Propeller Pumps Gasoline Engine Driven.
6 cfs. Vert. Cent. Pump Elect. Motor Driven Wet Well Type.
Sluice Gate
Ladder
3'x4' Sluice Gate.
16'-0"±
24'-0"±
Sump & Pump
Ladder
Removable Trash Rack
10" Dike Toe drain
18" C.I.
1-Length of C.I. Pipe
36" Storm Drain
Existing 24" Sanitary Sewer
1-Length C.I. Pipe
Inv. 17.5±
18"
NORTH ANDOVER PUMPING STATION
PUMP ROOM PLAN
SCALE 1/4" = 1'-0"
SCALE 1/4" = 1 FOOT
SUPERSEDED
MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION - SHAWSHEEN RIVER
LAWRENCE & NORTH ANDOVER, MASS.
LAYOUT OF PUMPING STATIONS
SCALES AS SHOWN
U. S. ENGINEER OFFICE, BOSTON, MASS.
JAN. 1945
APPROVAL RECOMMENDED
SUBMITTED
APPROVED
REVISIONS
FILE NO. M 55-12/25
PLATE 6-A.


WAR DEPARTMENT
CORPS OF ENGINEERS U. S. ARMY
ELEVATION IN FEET ABOVE M. S. L.
48
44
40
36
32
28
24
20
0
40
80
120
160
200
DISCHARGE IN MERRIMACK RIVER
1000 C. F. S.
1936
1938
Top of earth dikes
Top of concrete walls
1936 flood reduced by reservoirs
Low ground elevation in
North Andover & South Lawrence
MERRIMACK VALLEY FLOOD CONTROL
LAWRENCE AND NORTH ANDOVER
LOCAL PROTECTION
STAGE-DISCHARGE RATING CURVE
U.S. ENGINEER OFFICE, BOSTON, MASS. MAY, '44.
PLATE 9

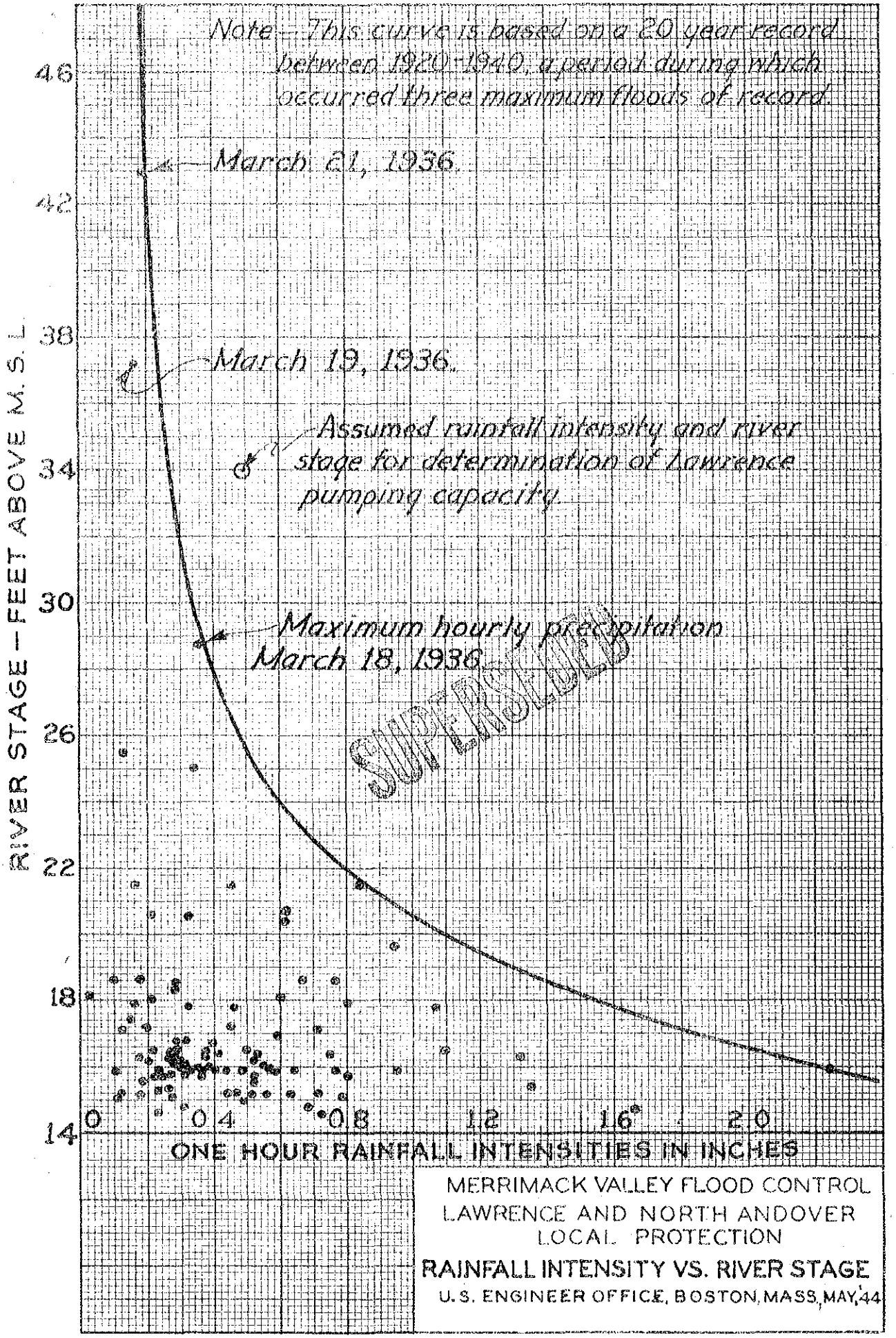


MERRIMACK VALLEY FLOOD CONTROL
LAWRENCE AND NORTH ANDOVER
LOCAL PROTECTION

STAGE – FREQUENCY CURVE

U. S. ENGINEER OFFICE, BOSTON, MASS., MAY, '44.

PLATE 10

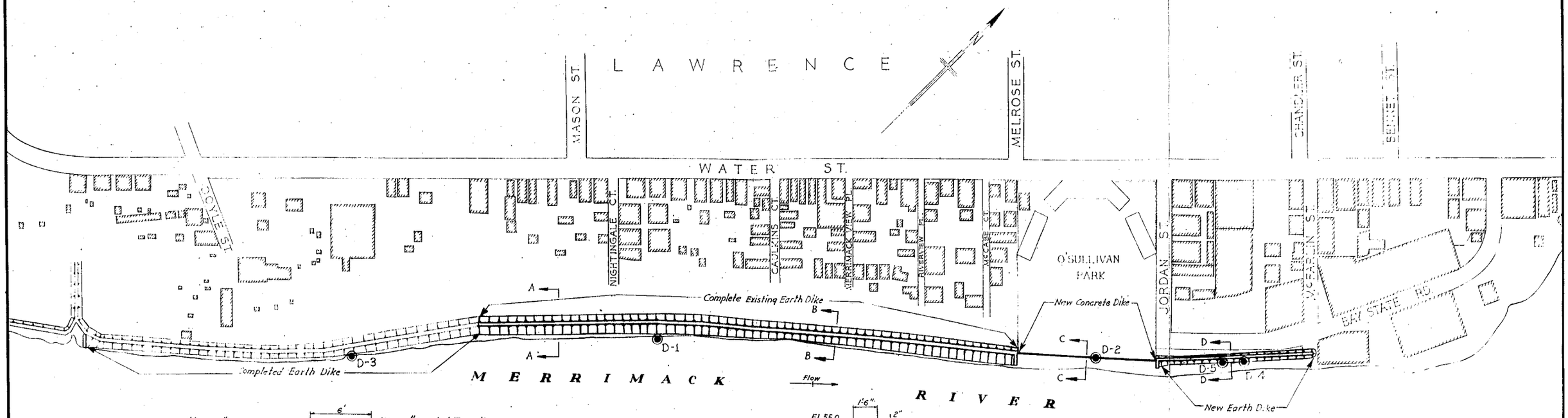


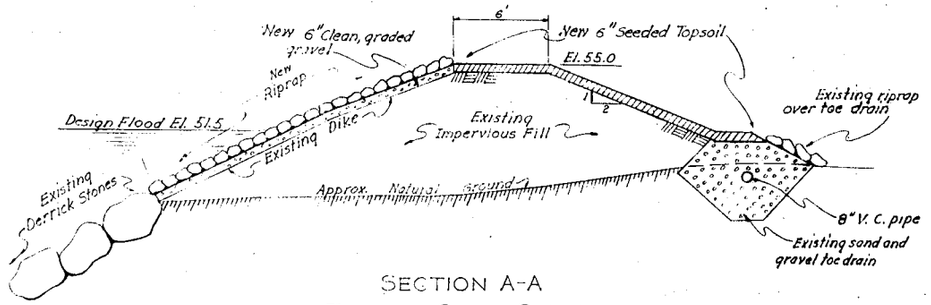
MERRIMACK VALLEY FLOOD CONTROL
LAWRENCE AND NORTH ANDOVER
LOCAL PROTECTION

RAINFALL INTENSITY VS. RIVER STAGE

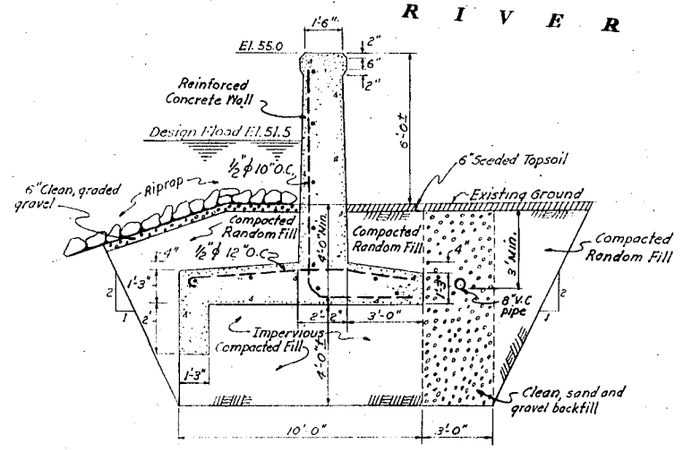
U. S. ENGINEER OFFICE, BOSTON, MASS., MAY, '44

PLATE II

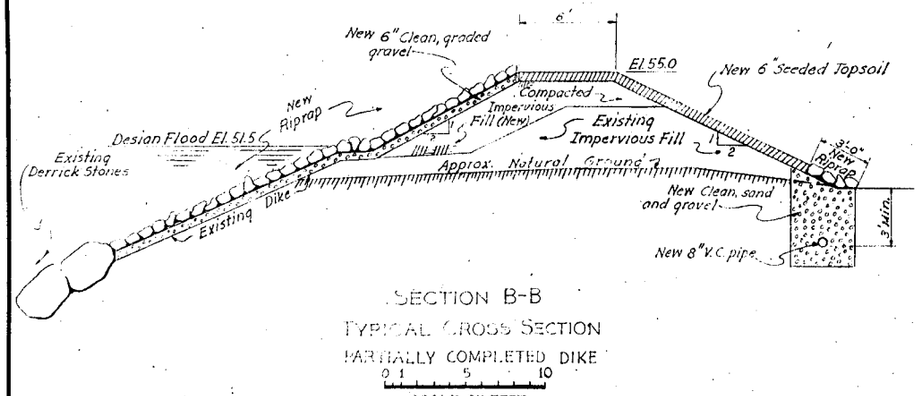




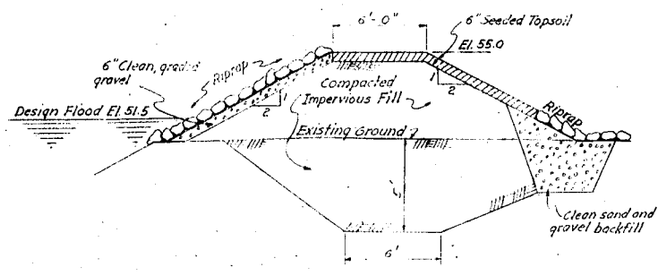
SECTION A-A
TYPICAL CROSS SECTION
PARTIALLY COMPLETED DIKE
0 1 5 10
SCALE OF FEET



SECTION C-C
TYPICAL CROSS SECTION
NEW CONCRETE DIKE
0 1 5 10
SCALE OF FEET



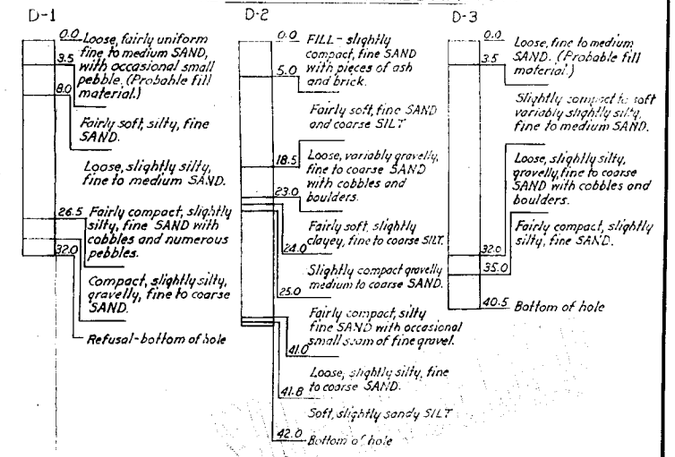
SECTION B-B
TYPICAL CROSS SECTION
PARTIALLY COMPLETED DIKE
0 1 5 10
SCALE OF FEET



SECTION D-D
TYPICAL CROSS SECTION
NEW EARTH DIKE
0 1 5 10
SCALE OF FEET

Note:- Indicated stage of the design flood is based on the March, 1936 Flood, reduced by the comprehensive reservoir system. Earth dikes and concrete walls are structurally designed with flood stage of top elevation.

RECORD OF EXPLORATION



MERRIMACK VALLEY FLOOD CONTROL
LOCAL PROTECTION-MERRIMACK RIVER
LAWRENCE, MASS.
PRELIMINARY LAYOUT
SCALE OF FEET
0 100 500

U. S. ENGINEER OFFICE, BOSTON, MASS. APRIL, 1944

APPROVAL RECOMMENDED
John E. Allen
CHIEF ENGINEERING DIVISION

APPROVED:
COLONEL, CORPS OF ENGINEERS
DISTRICT ENGINEER

SUBMITTED:
CHIEF DESIGN SECTION, EE ENGINEER

Ellen F. Chase
ENGINEER

DES. BY
DR. BY
TR. BY
CH. BY

FILE NO. M55-12/23

NO.	CHARACTER	DATE	BY

REVISIONS